GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Summary Report

FINAL REPORT

Prepared for

Greater Vancouver Regional District

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EXECUTIVE SUMMARY

The GVRD is embarking on a major facilities planning project for the Iona Island and Lions Gate wastewater treatment plants (IIWWTP and LGWWTP). The two plants provide preliminary and primary treatment of screening, grit removal, and primary sedimentation. Under the approved Liquid Waste Management Plan (LWMP), upgrading to full secondary treatment is required by 2020 at IIWWTP and by 2030 at LGWWTP. In the interim the plants must also reliably meet the following effluent requirements:

	Iona Island	Lions Gate
• Total Suspended Solids (TSS)	100 mg/L	130 mg/L
• Organic Matter (BOD_5)	130 mg/L	130 mg/L

An extensive review of sewage treatment processes was carried out in order to provide a short list of options for secondary treatment. The proposed options for secondary treatment and their estimated costs are summarized in Table 1.

TABLE 1
SUMMARY OF OPTIONS FOR SECONDARY TREATMENT

Plant	Options for Secondary Treatment	Estimated Capital Cost	Life Cycle Cost
Iona Island WWTP	Trickling Filter/Solids Contact (TF/SC)	$504,462,000	$281,022,000
	Biological Aerated Filters (BAF)	$457,905,000	$273,212,000
Lions Gate WWTP	Biological Aerated Filter (BAF)	$107,959,000	$35,444,000

Because of severe space constraints at the site of the Lions Gate plant, biological aerated filter (BAF) is the only recommended option for the long term since this process has the smallest footprint. For Iona Island, trickling filter/solids contact (TF/SC) is the recommended process. Since BAF is an option for Lions Gate and its cost is comparable to TF/SC, therefore, BAF process may also be suitable for Iona Island plant.

Forecasts of effluent quality for the two primary treatment plants were carried out. In order to maintain 99% reliability in meeting the above effluent criteria, interim upgrades must be carried out for Iona Island and Lions Gate depending on growth and increased flow and loading. The proposed options for interim upgrades and their estimated costs are summarized in Table 2. For Iona Island, option 2 (biological treatment for 50% of average dry weather flow, ADWF), option 3 (chemically enhanced primary treatment, CEP) and option 4 (CEP following by 50% ADWF biological treatment) would ensure reliability in meeting the effluent criteria until 2021. Option 1 (biological treatment for 25% of ADWF) would ensure reliability until 2016. For Lions Gate, all three interim options would ensure reliability in meeting the effluent criteria until 2031.

TABLE 2
SUMMARY OF OPTIONS FOR INTERIM UPGRADES

Plant	Options for Interim Upgrades	Estimated Capital Cost	Life Cycle Cost
Iona Island WWTP	Option 1: Biological treatment for 25% of average dry weather flow (ADWF) using roughing trickling filters (RTF)	$96,724,000	$102,002,000
	Option 2: Biological treatment for 50% of ADWF using RTF	$180,162,000	$173,268,000
	Option 3: Chemically enhanced primary treatment (CEP)	$97,480,000	$149,905,000
	Option 4: CEP followed by biological treatment for 50% of ADWF using RTF	$143,759,000	$184,810,000
Lions Gate WWTP	Option 1: CEP only	$25,756,000	$25,534,000
	Option 2: BAF for 50% of ADWF	$49,540,000	$35,834,000
	Option 3: CEP followed by BAF for 50% of ADWF	$56,571,000	$46,092,000

The proposed schedule for early interim upgrades and build-out to secondary at Iona Island is shown in Table 3. A four-cell lagoon is currently in use at Iona Island for sludge thickening and dewatering. Dewatered sludge is then stored on the portion of the site that will be required for plant expansion. The use of the lagoon and on-site sludge stockpiling will have to be gradually phased out between 2005 and 2016. Extensive site preparation is required at Iona Island including the removal of sludge stockpiles, pre-loading, and ground densification to prevent lateral movements during earthquakes.

TABLE 3
PROPOSED SCHEDULE FOR IONA ISLAND WWTP

	Interim Upgrades				Build-out to Secondary	
	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP	Option 4 CEP + 50%ADWF RTF	Option 1 TF/SC	Option 2 BAF
Decision Review	2005	2005	2005	2005	2015	2015
Design and Tender	2006-07	2006-07	2006-07	2006-07	2016-17	2016-17
Construction	2008-09	2008-09	2008-09	2008-09	2018-20	2018-20
Empty Sludge Storage Lagoon	2006 (Cell 1)	2006 (Cell 1)	2006 (Cell 1)	2006 (Cell 1)	2016 (Cells 2-4)	2016 (Cell 2-4)
Remove Sludge Stockpiles	2006	2006	2006	2006	2016	2016
Preloading	2007	2007	2007	2007	2017	2017

The proposed schedule for early interim upgrades and build-out to secondary at Lions Gate is shown in Table 4.

TABLE 4
PROPOSED SCHEDULE FOR LIONS GATE WWTP

	Interim Upgrades			Build-out to Secondary
	Option 1 CEP	Option 2 50% ADWF BAF	Option 3 CEP + 50% ADWF BAF	BAF
Design and Tender	2014	2014	2014	2026-2027
Construction	2015-16	2015-16	2015-16	2028-30

In addition to expanding the plant on the current site, several options were also examined for relocating the LGWWTP on other sites. These options are summarized in Table 5.

TABLE 5
SUMMARY OF OPTION FOR RELOCATING LIONS GATE PLANT

Option		Capital Cost	Remarks
1	Expansion on current site	$107,959,000	Plant located on leased land
2	Replace current plant with 3 smaller plants on the North Shore	$185,000,000	Three sites include: • Near Ambleside park • Current site • Lynn Valley pump station
3	Relocate the plant to another site on the North Shore	$160,000,000	Alternative site located within 1 km of existing site
4	Pump the sewage from the North Shore to Iona Island for treatment	$221,000,000	• Includes the cost of a twin marine pipeline from the current plant to the north end of the Highbury Interceptor ($58 M) • Includes the cost of additional primary and secondary treatment at Iona Island ($97 M) • Includes a wet weather plant on the North Shore ($66 M)

1.0 INTRODUCTION

1.1 BACKGROUND

Stantec Consulting Ltd. and Dayton & Knight Ltd. were retained by the Greater Vancouver Regional District (GVRD) to prepare facility plans for Iona Island and Lions Gate wastewater treatment plants (IIWWTP and LGWWTP). The study of geotechnical component was completed by Trow Associates as a sub-consultant to Stantec Consulting Ltd.

The GVRD is embarking on a major facilities planning project for the Iona Island and Lions Gate primary wastewater treatment plants. The facility plans were developed to satisfy the requirements of the Liquid Waste Management Plan for Iona Island and Lions Gate, to ensure permit requirements for 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) concentrations are satisfied and to examine options to reduce the non-ammonia acute toxicity in effluents. Reduction in non-ammonia toxicity would be required should the receiving environment monitoring program demonstrate there is an impact from the effluent discharge. Planning beyond the 20-year or 30-year horizon is also considered in the development of the facility plans to outline future requirements for secondary treatment.

The interim upgrades for permit compliance are designed to be in place until both plants are upgraded to secondary treatment plants. Under the approved Liquid Waste Management Plan, upgrading to full secondary treatment is required by 2021 for Iona Island WWTP and by 2031 at Lions Gate WWTP.

At the IIWWTP, there is concern that the maximum permitted discharge BOD_5 concentration could be reached before 2021. Options were investigated to identify alternatives to ensure plant permit compliance until the plant is upgraded to secondary treatment in 2021. Similarly at the LGWWTP, options were investigated to ensure permit compliance until 2031.

Monthly tests at both plants indicate some level of non-ammonia toxicity in samples taken prior to plant discharge. Previous studies have indicated that the most likely cause of effluent toxicity at Iona Island as measured by the 96 hour bioassay is related to a low level of dissolved oxygen (DO) and the toxicity at Lions Gate is related to surfactants.

Another issue for the Lions Gate plant is the limited land available at the site for both near-term upgrades and long-term needs to construct a secondary treatment plant.

As well as the facility upgrade, requirements need to take into account population growth over the next 50 years, changes in flow and concentration resulting from sewer separation and infiltration and inflow (I/I) reduction programs, and the existing condition of the plants. Both plants were built in several stages starting in the early sixties and several key components of the plants are aged and now over 40 years old.

1.2 SUMMARY AND SCOPE OF WORK

Given the large number of issues, the facilities planning study was broken up into several tasks. This breakdown has allowed the GVRD to involve the appropriate staff to provide inputs and advice to the Consulting team and facilitated the overall management of a study, which has a broad scope of work. The key tasks are:

<u>Task 1 – Domestic and Commercial Trucked Liquid Waste</u>

One of the main objectives of this task was to assess the Iona Island trucked liquid waste discharge to determine the impact of the trucked waste on effluent quality. The concern with the trucked liquid waste is that due to their high strength and discharge patterns, there could be significant impacts on effluent quality (BOD_5 and TSS) as well as effluent toxicity. This work is described in Appendix 1.

<u>Task 2 – Low Dissolved Oxygen in IIWWTP Sewerage Network Tributary</u>

The objective of this task was to develop sewer system options that would improve the dissolved oxygen levels in the influent flow to the Iona Island treatment plant. More specifically, options for in-sewer treatment were examined including in-sewer aeration in the Highbury tunnel, in-sewer chemical addition to chemically degrade organics or to suppress the growth of microorganisms. This work is described in Appendix 2.

<u>Task 3 – Interim Treatment Facility Upgrading Requirements</u>

Tasks 3 and 4 were the core of this project and covered a large number of topics. The main topics covered under Task 3 included the preparation of detailed flow and load projections for the Vancouver Sewage Area (VSA) and the North Shore Sewage Area (NSSA), as well as a description of the Iona Island and Lions Gate wastewater treatment plant condition.

For both plants, a comprehensive number of options for interim upgrades were identified. These include: (1) physical/chemical processes including chemically enhanced primary (CEP) treatment, (2) biological treatment for a portion of the average dry weather flow (ADWF), (3) combination of CEP and partial biological treatment, (4) dissolved air flotation and (5) combination of primary treatment and chemical oxidation. This was followed by an analysis of options and a two-step screening methodology in order to identify preferred options for interim upgrades for each plant. In this Task, alternative processes to conventional CEP (Task # 6) were also investigated in conjunction with other process options. This work is described in Appendix 3.

<u>Task 4 – Consideration for Build-out to Secondary</u>

For both plants, a comprehensive number of options for build-out to secondary were identified. These include: (1) fixed-film processes, (2) suspended growth processes, (3) anaerobic processes, (4) combination of fixed film and suspended growth (5) chemical oxidation, (6) primary treatment followed by partial biological treatment and (7) chemically enhanced primary (CEP) treatment followed by partial biological treatment. All the options were screened and ranked using a two-step screening approach. Preferred options for build-out to secondary were identified. This work is described in Appendix 4.

Task 5 – Small Scale Testing Program

The initial scope of work for this task included the evaluation of options for chemically enhanced primary (CEP) treatment for interim treatment and small-scale testing. The work dealing with CEP evaluation was combined with Task # 3 and the work carried out under Task # 5 consisted of the small scale testing program only. The small scale testing program was mainly a treatability study to determine treatment processes that could significantly reduce the frequency of failure to pass the 96 hours LC_{50} toxicity bioassay at both plants. This work is described in Appendix 5.

Task 6 – Alternatives to Conventional CEP

This task was combined with Task # 3.

Task 7 – North Shore Sewage Flow Diversion

This task involved developing conceptual plans based on the possible diversion of flows from the North Shore to the IIWWTP. The feasibility of a marine pipeline crossing with a pumping facility located at the existing Lions Gate plant and alternate North Shore location for a treatment plant were assessed. An examination of a range of flow diversion scenarios and the North Shore wet weather options are reviewed, from full diversion to diversion of only dry weather flow. This work is described in Appendix 6.

Task 8 – Interim Solids Handling

This task included the assessment of current sludge handling unit processes, including sludge thickening, sludge stabilization, and dewatering. This was followed by the prediction of sludge quality and quantity of interim and build-out to secondary process upgrades identified in Task # 3 and Task # 4. Potential treatment technologies to reduce sludge volume and improve sludge quality were identified and interim treatment options to handle sludge were recommended. This work is described in Appendix 7.

Task 9 - Current Condition of Existing Plants

The objective of this task is to evaluate the general condition of the various unit processes, tanks and major equipment in order to determine if a component should be replaced in order to meet the following two conditions (1) to integrate the existing primary plant with the proposed secondary plant, and (2) to ensure that the treatment facility can be operated satisfactorily for the next 50 years. The purpose of this task is not to duplicate the existing operating and maintenance schedule of the plants. This work is described in Appendix 8.

Task 10 – Geotechnical Assessments

A preliminary geotechnical assessment was carried out for both plants and included review of the subsoil conditions, foundations, seismicity and potential rise in sea level. Preliminary assessment and recommendations for the proposed pipeline routing from Lions Gate plant to Iona Island were also provided. This work is described in Appendix 9.

Task 11 – Life-Cycle Capital and Operating Costs and Sensitivity Analysis

Life cycle capital and operating/maintenance costs were developed for a number of options for both interim upgrades and build-out to secondary. These were presented in the appendices produced for Tasks # 3 and #4, respectively. Revised cost estimates for the preferred options were prepared and are presented in Appendix 10.

Task 12 – Engineering Pre-design Drawings and Specifications

Pre-design drawings were prepared for the preferred options identified in Tasks # 3 and # 4. Appendix 10 was also prepared to present the detailed analysis of the preferred options for interim upgrade and build-out to secondary. The short list of preferred options was identified at the end of Appendices #3 and # 4 but the analysis of this short list of options is included in Appendix 10.

Appendix 10 also included the following:

- Revised flow and load projections,
- Estimated sludge volumes for the preferred options,
- Forecast of effluent quality and permit compliance levels, and
- Revised capital, life cycle cost, and O&M costs.

Summary Report

This summary report integrates and summarizes the findings of the twelve tasks described above.

1.3 FORMAT OF THE REPORT

As indicated in Section 1.2 above, the project was divided into twelve key tasks. Ten separate appendices were issued detailing the work carried out under Tasks #1 to # 12. These technical memorandums are included as Appendix # 1 to # 10 to this report and are issued in separate volumes (Volume 2 ~ Volume 5). These Appendices are listed in Table 1.1.

TABLE 1.1
TECHNICAL APPENDICES TO THE SUMMARY REPORT

Volume #	Appendix #	Task #	Document Title
2	1	Task 1	Domestic and Non-Domestic Trucked Liquid Waste
	2	Task 2	Low Dissolved Oxygen in Iona Island WWTP Tributary Network
	3	Tasks 3 & 6	Interim Facility Upgrading Requirements
3	4	Tasks 4 & 11	Consideration of Build-out to Secondary Treatment
	5	Task 5	Results of Small Scale Testing Program
	6	Task 7	Diversion of North Shore to Iona Island
4	7	Task 8	Interim Solids Handling Facilities
	8	Task 9	Current Condition of Treatment Plants
	9	Task 10	Geotechnical Assessment - Trow Consulting Ltd.
	10	Task 11 & 12	Analysis of Preferred Options
5		Task 12	Preliminary Design Drawings

The purpose of this Summary Report is to integrate and summarize the findings of the study, and is meant to be a stand-alone document for those readers who do not need the level of details found in the Appendices. The various sections of this report are grouped under several parts as follows:

- Executive Summary
- Introduction (Section 1)
- Part 1 – Upgrading of Iona Island WWTP (Section 2 to Section 6)
- Part 2 – Upgrading of Lions Gate WWTP (Section 7 to Section 11)
- Part 3 – North Shore Sewage Treatment Alternatives (Section12 to Section 14)

Each section within the Summary Report is based on the findings presented in various appendices. Table 1.2 lists the major sections for Part 1, 2, and 3, in which the detailed information covered can be located in various Appendices.

TABLE 1.2
SUMMARY REPORT SECTION CROSS REFERENCE TO APPENDICES

Summary Report Sections	Appendix #
2.1 – 2.3	3
2.2 - 2.5	10
3.1	1
3.2	2
3.3	5
3.4	8
3.5	9
3.6	8
4.1 – 4.4	4
4.5 – 4.6	10
5.1 - 5.4	3
5.5 - 5.8	10
6.1	7, 10
6.2 - 6.5	7
6.6	10
7.1	3
7.2 - 7.5	10
8.1	5
8.2	8
8.3 - 8.4	9
8.5	4
8.6	8
9.1 - 9.4	4
10.1 - 10.7	3
10.8 – 10.9	10
11.1	7, 10
11.2 - 11.4	7
12.1 - 12.3	4
13.1 - 13.5	4, 6

Task 12 also included the preparation of conceptual drawings and specifications for the preferred options. These drawings are found under separate cover (Volume 5, Interim and Build-out to Secondary Stage, Preliminary Design Drawings). The list of drawings for Iona Island and Lions Gate plants are included in Tables 1.3 and 1.4 respectively.

TABLE 1.3
LIST OF PLANS FOR IONA ISLAND WWTP

Sheet	Title	Description
1	Title Sheet	
2	Process Schematic	Existing Primary Plant and TF/SC Option
3	Overall Site Plan	Existing Primary Plant and TF/SC Option
4	Plant Layout – Liquid Stream	Trickling Filters; Solids Contact; Secondary Clarifiers; Flow Separation and Pumping
5	Plant Layout – Solids Stream	DAF, Digesters, Dewatering, Sludge Blending Tanks
6	Conceptual Hydraulic Profile	Hydraulic Profile For Liquid Stream For Secondary Plant
7	Conceptual Hydraulic Profile – Solid Stream	Hydraulic Profile For Solids Stream For Secondary Plant
8	Trickling Filter (TF) - Typical Unit	Plan And Sections
9	Solids Contact (SC) Tank – Typical Unit	Plan And Sections
10	Secondary Clarifier – Typical Unit	Plan And Section
11	Anaerobic Digester – Typical Unit	Plan And Section
12	Dissolved Air Flotation (DAF) – Typical Unit	Plan And Sections
13	Centrifuge Building	Plan And Sections
14	Chemical Feed Building	Plan And Section
15	Process Piping - Liquid Stream	Piping Interconnection Overlaid on Plant Layout
16	Process Piping – Solids Stream	Piping Interconnection Overlaid on Plant Layout
17	Biological Aerated Filter	Layout Plan

TABLE 1.4
LIST OF PLANS FOR LIONS GATE WWTP

Sheet	Title	Description
1	Title sheet	
2	Process Schematic	Existing Primary Plant and BAF Option
3	Summary of Design Criteria	
4	Hydraulic Profile	
5	Chemically Enhanced Treatment Till 2031	Layout Plan
6	Partial Biological Treatment Till 2031	Layout Plan
7	Biological Aerated Filters	Layout Plan and 3D Drawing
8	DAF	Plan and Section
9	Digesters 5 and 6	Plan and Section
10	Chemical Feed Room and Alum Storage Tanks	Plan and Section
11	Interconnecting Piping	

PART 1 – UPGRADING OF IONA ISLAND WWTP

2.0 ANALYSIS AND EVALUATION CRITERIA

2.1 EFFLUENT CRITERIA

2.1.1 Liquid Waste Management Plan Requirements

The Liquid Waste Management Plan has stipulated that the base level of treatment for Iona Island treatment plant should meet the following maximum daily concentration levels as indicated in the Operational Certificate ME-00023:

- BOD_5 130 mg/L
- TSS 100 mg/L

The above concentrations are based on flow proportional 24-hr composite samples. The Liquid Waste Management Plan (LWMP) further indicates that the Iona Island Treatment plant will provide primary treatment for flows up to 17 m^3/sec (1,469 MLD). One of the commitments of the LWMP is to upgrade the plant by adding facilities for chemical addition if necessary in order to meet the above effluent concentrations. Another commitment of the LWMP is to upgrade the level of treatment if the base level of treatment is not adequate to protect the aquatic environment.

2.1.2 Municipal Sewage Regulation

The Liquid Waste Management Plan does not contain a definition of secondary treatment for the Iona Island plant nor does it include effluent criteria. The ***BC Municipal Sewage Regulation*** includes the following definition of secondary treatment:

Secondary treatment – any form of treatment, excluding dilution, that consistently produces an effluent quality with a BOD_5 not exceeding 45 mg/L and TSS not exceeding 45 mg/L for flows up to 2.0 x ADWF.

2.2 FLOWS AND LOAD SCENARIOS

2.2.1 Methodology and Definitions

Detailed flow and load projections were carried out in order to generate a lower and upper envelope as well as a most probable upper case scenario. Separate flow and load projections were prepared for the various contributors (dischargers) to the sewer system. The five contributors are:

(1) Residential
(2) Commercial and institutional (C&I)
(3) Industrial
(4) Groundwater infiltration and surface runoff
(5) Trucked liquid waste (TLW).

For each contributor, lower and upper growth rates were established and the impact of various scenarios for source control were estimated. Lower and upper envelopes for flows and loads at the plant were prepared by adding the lower and upper envelopes for all five contributors.

Following a review of the upper and lower envelopes, a design case was added to the flow and loads projections since it is unlikely that all the assumptions used to establish the upper and lower envelopes would occur at the same time.

It would be useful to recapitulate the definition of key terms used in loads and flow projections for Iona Island wastewater treatment plant.

Average Dry Weather Flow (ADWF). During period of little or no precipitation, wastewater flow is composed primarily of sanitary sewage, industrial and commercial wastes, and base infiltration. The ADWF is the design factor used to evaluate the performance of biological treatment during the warm summer months. ADWF is determined by sorting daily flow for a one-year period and selecting the 25^{th} percentile value.

Average Annual Flow (AAF). This is defined as the wastewater flow averaged over a calendar year. For Iona Island, the average annual flow is 1.34 x ADWF

Maximum Month Load (MML). This is defined as the maximum 30-day period based on an analysis of load data. This does not necessarily correspond to a calendar month. The MML is used in the sizing of the biological treatment units. For the period of 1991 to 1999, the MML peaking factors are 1.31 and 1.38 for BOD and TSS respectively.

<u>Population</u>

Population forecast is one of the basic components to establish sewer flows and load forecast. Population scenarios are shown in Table 2.1.

TABLE 2.1
POPULATION SCENARIOS FOR IIWWTP

Year	Lower Envelope	Upper Envelope	Design Case
2001– Census	616,379		
2021	700,000	750,000	740,000
2036	710,000	775,000	762,000
2051	720,000	800,000	784,000

2.2.2 Impact of Water Conservation Programs

In conjunction with the variability in population growth, the other significant factor in estimating future flows is the impact of water conservations measures. The impact of the existing water conservation program is taken into account for the most probable upper case scenario while the upper envelope assumes that per capita sewage generation rates would remain mostly unchanged. The per capita flows for various scenarios for residential and commercial & Institutional are indicated in Table 2.2.

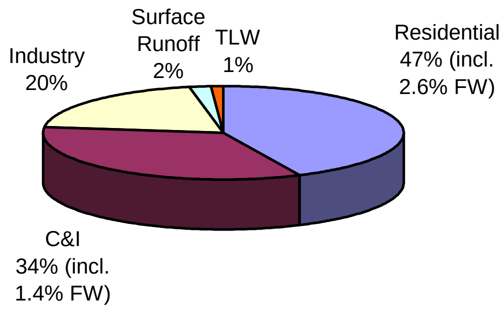
TABLE 2.2
RESIDENTIAL AND COMMERCIAL PER CAPITA FLOWS FOR IIWWTP

Year	**Lower Envelope (L/c/d)**	**Upper Envelope (L/c/d)**	**Design Case (L/c/d)**
2001 – Existing All sources	• Residential (Res.): 270 • Commercial (Com.): 166		
2021	• Res.: 214 • Com: 153	• Res.: 270 • Com: 166	• Res.: 220 • Com: 166
2036	• Res.: 175 • Com: 144	• Res.: 270 • Com: 166	• Res.: 188 • Com: 166

2.2.3 BOD and TSS Loading

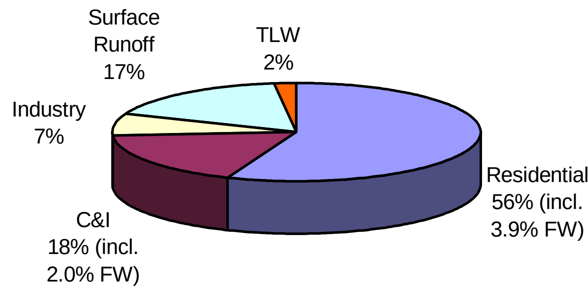
The existing BOD and TSS contributions from the various sectors are shown schematically in Figures 2.1 and 2.2. Water conservation measures will have no impact on loading. The only variable affecting loading for the residential and the commercial and institutional (C&I) sectors is the contribution from the food garburators. Tables 2.3 to 2.6 summarize the BOD and TSS loadings from the various sectors.

FIGURE 2.1
BOD LOADING (2002) FOR IIWWTP



Total BOD (Annual Average) = 74.5 tonnes/day

FIGURE 2.2
TSS LOADING (2002) FOR IIWWTP



Total TSS (Annual Average) = 70.0 tonnes/day

Note: Food Waste (FW)

TABLE 2.3
RESIDENTIAL AND C&I BOD FOR IIWWTP (ANNUAL AVERAGE)

Year	Lower Envelope (g/c/d)	Upper Envelope (g/c/d)	Design Case (g/c/d)
2001 – Existing All sources	• Residential (Res.): 53 • Commercial (Com.): 41		
2021	• Res.: 52 • Com: 39	• Res.: 54 • Com: 41	• Res.: 54 • Com: 41
2036	• Res.: 51 • Com: 36.6	• Res.: 54.6 • Com: 41	• Res.: 54.6 • Com: 41

Note: Multiply all values by 1.31 to obtain the maximum month for BOD

TABLE 2.4
INDUSTRIAL, TLW AND RUNOFF BOD FOR IIWWTP (MAXIMUM MONTH)

Year	Lower Envelope (t/d)	Upper Envelope (t/d)	Design Case (t/d)
2001 – Existing All sources	• Industrial (Ind.): 23.6 • TLW: 2.1 • Runoff: 1.8		
2021	• Ind. 22.6 • TLW: 2.2 • Runoff:1.8	• Ind. 28.3 • TLW: 2.5 • Runoff: 1.9	• Ind. 27.6 • TLW: 2.5 • Runoff: 1.9
2036	• Ind. 22.7 • TLW: 2.2 • Runoff: 1.8	• Ind.: 29.2 • TLW: 2.6 • Runoff: 2.0	• Ind.: 28.3 • TLW: 2.6 • Runoff: 2.0

TABLE 2.5
RESIDENTIAL AND C&I TSS FOR IIWWTP (ANNUAL AVERAGE)

Year	Lower Envelope (g/c/d)	Upper Envelope (g/c/d)	Design Case (g/c/d)
2001 – Existing All sources	• Residential (Res.): 61 g/c/d • Commercial (Com.): 21 g/c/d		
2021	• Res.: 59 • Com: 20	• Res.: 62 • Com: 21	• Res.: 62 • Com: 21
2036	• Res.: 59 • Com: 19.6	• Res.: 63 • Com: 21	• Res.: 63 • Com: 21

Note: Multiply by 1.38 to obtain maximum month.

TABLE 2.6
INDUSTRIAL, TLW AND RUNOFF TSS FOR IIWWTP (MAXIMUM MONTH)

Year	Lower Envelope (t/d)	Upper Envelope (t/d)	Design Case (t/d)
2001 – Existing All sources	• Industrial (Ind.): 6.8 t//d • TLW: 5.9 t/d • Runoff: 15 t/d		
2021	• Ind.: 7.0 • TLW: 5.9 • Runoff: 15	• Ind.: 8.6 • TLW: 7.1 • Runoff: 16	• Ind. : 8.4 • TLW: 7.0 • Runoff: 16
2036	• Ind. : 7.3 • TLW: 6.0 • Runoff: 15	• Ind.: 8.9 • TLW: 7.3 • Runoff: 17	• Ind.: 8.6 • TLW: 7.2 • Runoff: 17

2.3 ANALYTICAL ASSUMPTIONS

The flow and load parameters for the lower and upper projection envelopes are based on the following assumptions:

(1) Population projections are estimated in accordance with the population of upper and lower ranges developed by the GVRD's Regional Development Division for the VSA as indicated in Table 2.2 above.

(2) The upper envelope for flow is based on no new source control measures and an increase of infiltration by 5%. The upper envelope did not take into account the City of Vancouver 1994 requirements for low flush toilets. The lower envelope for flow is based on enhanced scenario for water conservation and a 10% decrease in infiltration as a result of the sewer separation program.

(3) The lower and upper growth rates for commercial and institutional flows and loads, as well as for the trucked liquid waste, are based on the same growth rates as population.

(4) The upper and lower projection of residential and C&I load are based on food waste discharge. Upper envelope of residential load assumes 80% of new households would be equipped with food grinders, while lower envelope assumes food grinders in residential households are reduced from one third of all households to 10% of all households in the design year.

(5) The lower envelope for industrial flows and loads is based on a growth rate that is similar to the lower range for population projection. However, the existing industrial sector is assumed to grow at 50% of population growth. The upper envelope for industrial flows and load is based on upper growth rate for population

Flow and load parameters developed in this study for the design case projection are based on the following assumptions:

(1) Population projection is estimated at 80% of the difference between the upper and lower ranges as indicated in Table 2.2.

(2) Flow projection from groundwater infiltration source is estimated at 80% of the difference between the upper and lower envelopes.

(3) Increases in commercial and institutional flows and loads are based on the same growth rates as population.

(4) Residential and C&I loads assume the same growth rate as the upper envelope.

(5) Per capita residential flow of 220 L/cap/day based on existing water conservation measures.

(6) Industrial flows and loads are the same as upper envelope

It is projected that the combination of lower population growth rates after 2021 coupled with the impact of existing water conservation measures will result in a 3% decrease for the ADWF for the design case for the period 2021-2036. However, since water conservation measures do not have an impact on the loading, the TSS and BOD for the design case increase by 2% during the same period.

It is also assumed that the peak wet weather flow will remain the same over the period to 2003-2036 since the sewer separation program will take over 50 years to complete. As stormwater flows are eliminated from one area, the additional capacity resulting from that reduction will be used to intercept combined sewage that may otherwise have overflowed the sewer system.

2.4 PROPOSED ANALYTICAL SCENARIOS AND CRITERIA

2.4.1 Analytical Scenarios

Based on the data presented in Tables 2.1 to 2.6 above, the flow and load projections for various scenarios are summarized in Table 2.7. The projected flows and loads together with the existing data for the period 1991-2002 are shown graphically in Figures 2.3 to 2.5.

TABLE 2.7
FLOW AND LOAD SCENARIOS FOR IIWWTP

	2021 – Design Year for Interim Upgrades			2036 – Design Year for Build-out to Secondary		
	Lower Envelope	Upper Envelope	Design Case	Lower Envelope	Upper Envelope	Design Case
ADWF (Ml/d)	412	498	456	383	511	441
PWWF (Ml/d)	1530	1530	1530	1530	1530	1530
Max Month BOD (t/d)	108	127	124	108	131	127
Max Month TSS (t/d)	106	120	116	105	124	119

FIGURE 2.3
IONA ISLAND WWTP - FLOW PROJECTIONS

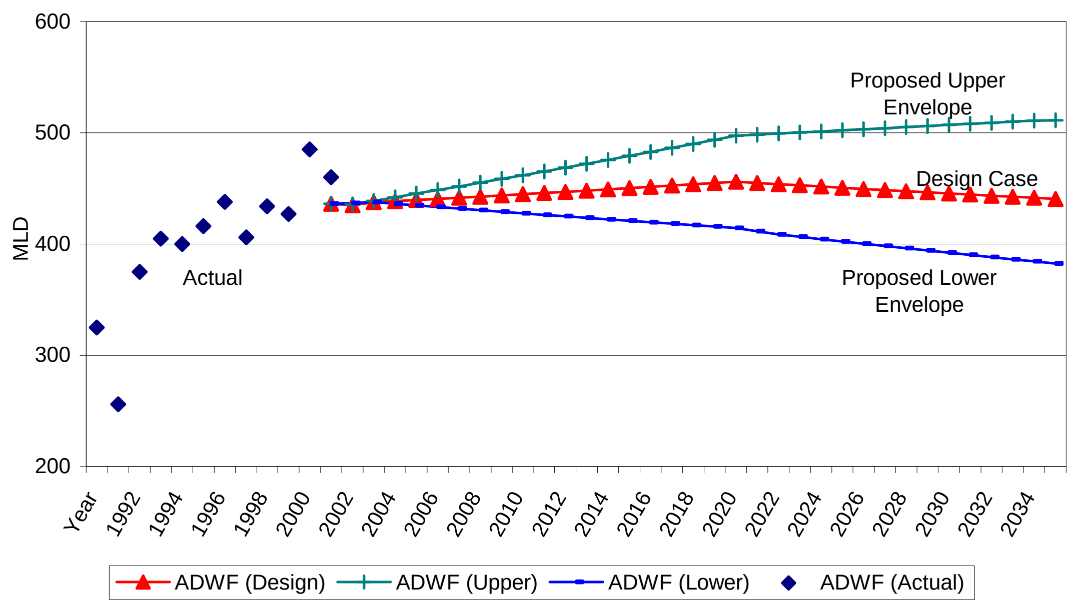


FIGURE 2.4
MAX. MONTH BOD PROJECTIONS (TLW INCLUDED)

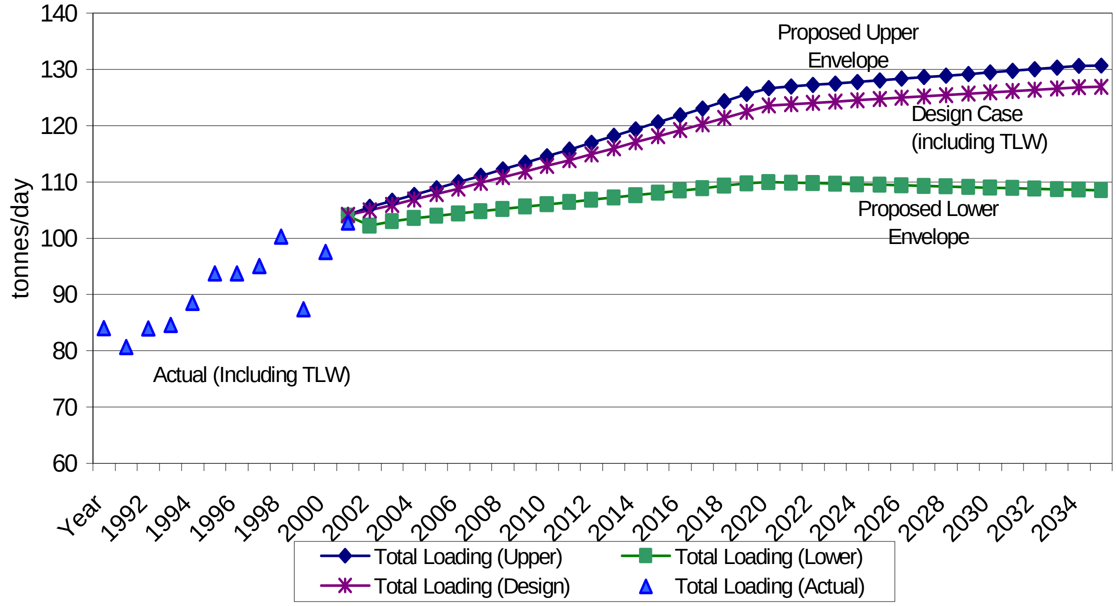
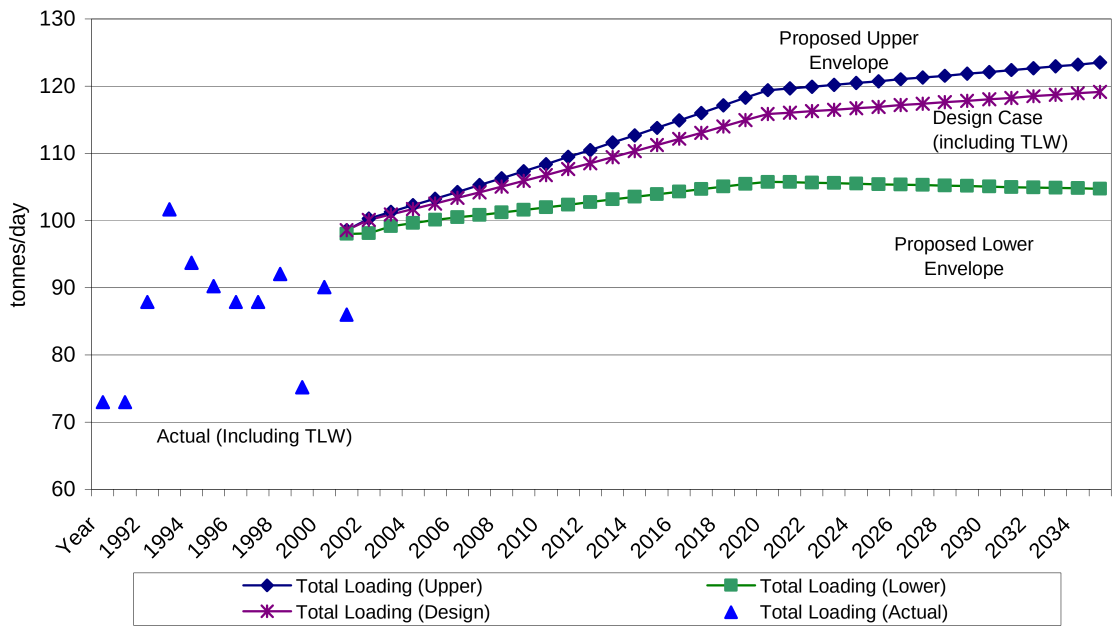


FIGURE 2.5
MAX. MONTH TSS PROJECTIONS (TLW INCLUDED)



2.4.2 Proposed Design Criteria

The analytical flows and loading for various scenarios are indicated in Table 2.7 above. The difference in the flows and loads between the year 2021 and 2036 are negligible. It is proposed to use the year 2021 flows for both the interim upgrades and build-out to secondary, since the plant has to be capable of dealing with the higher flow value. It is proposed to use the year 2036 loads for both the interim upgrades and build-out to secondary, since the plant must also be capable of dealing with the higher loads.

The proposed design criteria for the treatment plant upgrades are indicated in Table 2.8.

TABLE 2.8
PROPOSED DESIGN FLOWS AND LOADING FOR IIWWTP

Parameter	Interim Upgrades	Secondary Treatment
Design year	2021	2036
Design Flow to Primary Treatment (Ml/d)	1530	1530
Design Flow to Secondary Treatment (Ml/d)	225 (50% of ADWF)	912 (2 x ADWF)
Effluent Standard for BOD (mg/L)	130	45 (20*)
Effluent Standard for TSS (mg/L)	100	45 (20*)

* Design target

The effluent criteria for BOD_5 and TSS of 45 mg/L indicated in the Municipal Sewage Regulation are maximum limits never to be exceeded. In order to meet the maximum effluent criteria of 45/45 mg/L, design effluent criteria of 20/20 mg/L have been used when sizing the facilities.

2.5 PERMIT COMPLIANCE ISSUES TO 2021

2.5.1 Summary of Compliance Issues

For the interim period until 2021, there are two separate issues regarding effluent quality:

1. Permit Compliance for TSS (100 mg/L) and BOD_5 (130 mg/L)
2. Effluent toxicity - as measured by the 96-hour LC_{50} rainbow trout bioassay

The scheduling of interim upgrades is a function of which problem is being resolved. If the intent of the interim upgrade is to reduce effluent toxicity, the scheduling would be carried out based on discussions with the Regulatory Agencies following the assessment of the impact of effluent toxicity on the receiving environment.

However, as the average dry weather flow increases, the flow and loading to the primary plant will increase and the effluent quality will start to exceed the permit. The forecast of effluent quality for permit compliance is discussed in Section 5.6 of this report.

2.5.2 Primary Sedimentation Tank (PST)

Primary sedimentation tanks (PST) are operated to remove substantial portions of readily settleable solids and organic substrates associated with solids. An efficient PST system is capable of removing 50~70% of total suspended solids (TSS) and 25~45% of biochemical oxygen demand (BOD). However, the removal efficiency is subject to many factors, and many of these factors could be combined.

In addition, some specific operating conditions upstream of the PST will also affect the PST performance. At Iona Island these specific factors include flow distribution and influent pump operation. At the IIWWTP, the hydraulic factors (flow distribution, PST surface overflow rate and hydraulic retention time etc.) and wastewater characteristics (settleable TSS and organic content distribution etc.) are considered to be the most important factors affecting the PST performance.

2.5.3 Analysis of PST Performance

The effluent quality for TSS and BOD concentrations for the past ten years are shown in Figure 2.6 and Figure 2.7, respectively, against the percentile of occurrences. As a results of using all of the primary sedimentation tanks for wastewater treatment (prior to 1997, some PST were used only for TLW) the compliances level for effluent quality have improved substantially, from 90% (in 1990s) to 99% (in early 2000s) of TSS and 88% (in 1990s) to 98% (in early 2000s) of BOD, respectively, regardless of the flow and load increases through the years.

FIGURE 2.6
EFFLUENT TSS CONCENTRATIONS AT IIWWTP (1991~2002)

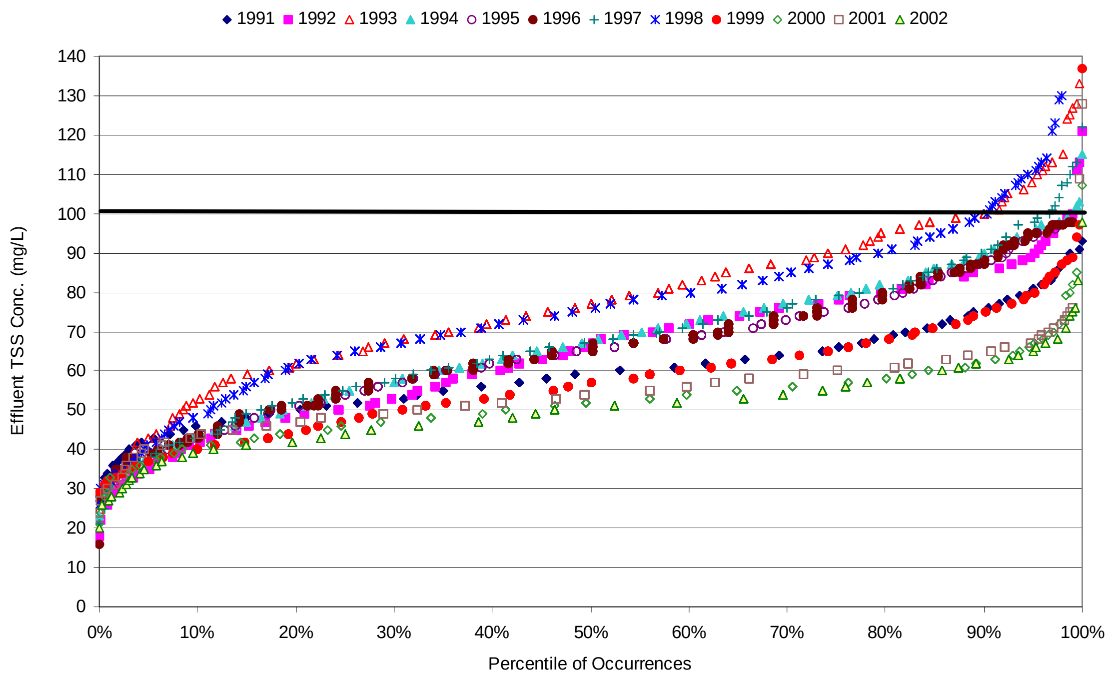
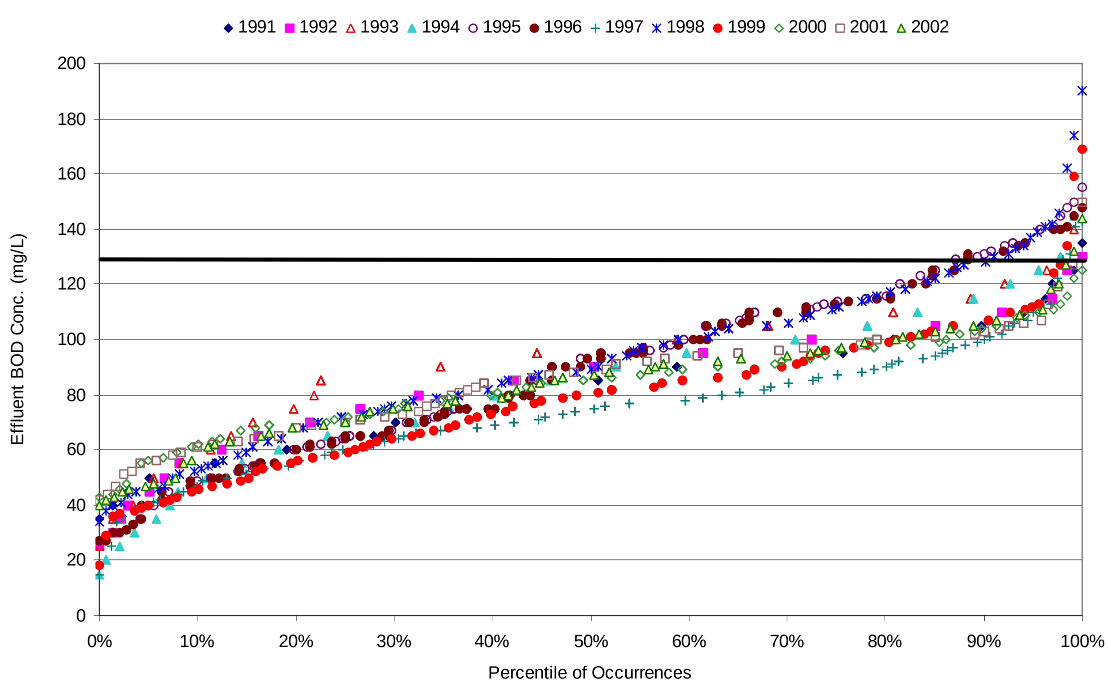


FIGURE 2.7
EFFLUENT BOD CONCENTRATIONS AT IIWWTP (1991~2002)



3.0 SUMMARY OF TECHNICAL STUDIES

3.1 TRUCKED LIQUID WASTE

Trucked liquid waste (TLW) generated in the Greater Vancouver Regional District are delivered to the Iona Island WWTP (IIWWTP), and Annacis Island WWT (AIWWTP) for treatment. The TLW are classified into two categories: domestic and non-domestic waste. Domestic TLW is collected from septic tanks, holding tanks, portable toilets and other domestic sources. Non-domestic waste is generated at industrial plants, agricultural operations and restaurant grease dumps. At present IIWWTP receives approximately 80% of the domestic TLW and 100% of the non-domestic TWL generated in the region. The remaining domestic TLW is received at the AIWWTP while the Northwest Langley WWTP (NWLWWTP) receives domestic TLW only.

At the IIWWTP, non-domestic TLW is pretreated in a rotary drum screen and clarifier. It should be noted that the supernatant from the scum thickener is also pumped to the TLW clarifier. The TLW clarifier supernatant is pumped to the influent channel of the primary sedimentation tanks #11 through #13. Domestic TLW is discharged directly into the influent siphon pipes prior to entering the plant headworks. AIWWTP is equipped with a septic receiving station to pre-treat the domestic TWL using a semi-cylindrical coil bar screen prior to mixing with the plant sewage influent.

The impact of the TLW on the effluent quality at the IIWWTP needed to be confirmed. In order to determine if the TLW is causing deterioration of the effluent quality (TSS and BOD concentrations), a mass balance was carried out to estimate the increase in effluent BOD/SBOD and TSS concentrations at the plant effluent resulting from these TLW discharges. A sampling program was conducted in the summer 2003 to obtain recent data on TLW characteristics and to compare it with sampling carried out in 1997. The objectives of the TLW sampling and mass balance analysis include:

- Determine the TLW quality and quantity,
- Determine the non-domestic TLW pre-treatment efficiency
- Assess the impacts of the TLW on the main treatment system.

The volume-weighted averages of BOD, SBOD and TSS concentrations are summarized in Table 3.1. Results from the 2003 sampling program indicated that the BOD/SBOD and TSS concentrations of the non-domestic TLW were almost double the concentrations measured in the 1997 sampling. The TSS concentration of the domestic TLW were also double the concentration recorded in the 1997 analysis.

TABLE 3.1
TLW CONCENTRATIONS FOR 1997 AND 2003 (VOLUME-WEIGHTED AVERAGES)

	1997 Sampling			2003 Sampling		
Parameters	BOD mg/L	SBOD mg/L	TSS mg/L	BOD mg/L	SBOD mg/L	TSS mg/L
Domestic TLW	1,560	550	5,060	1,570	460	10,520
Non-Domestic TLW	41,000	14,500	101,000	121,750	21,460	210,640

The removal efficiency of the IIWWTP non-domestic TLW pre-treatment process was estimated to be between 20 and 80% for BOD, negligible for SBOD, and 30 to 60% for TSS. The TSS removal efficiency based on the weekly samples was estimated about 45%. The pre-treatment efficiency was highly dependent on the waste characteristics, such as the settleable solids fraction, settleable BOD fraction, and organic degradation rates, and TLW discharge volumes. It should be noted that the PST scum flow may have a significant dilution effect.

The mass balance analysis indicated that the impact of TLW on the IIWWTP effluent was limited. The TLW could cause TSS concentration increases by 2 to 8 mg/L in the effluent composite samples. BOD and SBOD concentrations were increased by 1 to 2 mg/L, which was considered a marginal percentage of the overall discharge. The increase in TSS concentrations is expected to have a limited impact on the effluent toxicity, since previous studies have identified that elevated BOD/SBOD concentrations are the primary cause of toxicity in the effluent.

The final aspect of the TLW study was a questionnaire filled out by the haulers. It was found that the haulers were satisfied with two disposal sites (IIWWTP and AIWWTP). Longer operational hours, up to 24 hr per day, 7-days a week, would be beneficial to their operation. Queuing for discharge did not appear to be a major problem. Hauling distance, traffic conditions and operational hours were cited as the primary factors in the selection of which discharge station to use.

The results of the TLW study indicate that there is no immediate need to upgrade the existing treatment facilities. Operational changes to maximize the use of the existing treatment facilities could be considered the most appropriate planning strategies. These changes include:

- Expanding operating hours
- Add flow equalization in order to minimize overloading of primary clarifiers
- Diversion of domestic TLW to Annacis Island to maximize use of secondary treatment for trucked liquid waste.
- Continue with source control and monitoring

At the IIWWTP, the following minor upgrades are recommended including: (1) installing a septic receiving station at IIWWTP to serve the domestic TLW discharge, and (2) re-rating the rotary drum screen capacity and connecting piping. At the

AIWWTP, an upgrade to enlarge the existing grit/stone trap would reduce the maintenance frequency. With respect to future expansion plans, it is important to consider the TLW flows and loads at the IIWWTP for the treatment capacity and facility planning.

3.2 LOW DISSOLVED OXYGEN IN SEWER

It was postulated that toxicity reduction at IIWWTP could be influenced by corrective actions taken in the sewer collection system. The effluent toxicity issue at Iona Island has been identified as being caused primarily by oxygen depletion occurring during the monitoring toxicity testing. Oxygen depletion during the test is caused by the presence of readily degradable organics in the primary effluent as well as the action of microorganisms present in the primary effluent. GVRD personnel have noted the presence of organisms, similar to activated sludge organisms, in the samples sent to the toxicity-testing laboratory at times when the toxicity tests have failed.

Strategies to eliminate and reduce the occurrence of toxicity test failures by reducing the source of soluble organics and that have been proposed and described hereafter:

- Aeration of a portion of Highbury Tunnel and stimulation of aerobic biological growth and degradation of soluble organics.
- Addition of oxidizing chemicals into the sewer system such as hydrogen peroxide to chemically degrade organics.
- Suppression of the growth of microorganisms in the sewers so that their population in the primary effluent is sufficiently low, such that their respiration during the toxicity tests does not deplete the DO to a level that kill the test fish.
- Addition of a strong oxidizing agent to the sewers such as chlorine to suppress the microorganism activity.
- Changing the flow regime and eliminating sludge accumulations.

If such strategies were feasible, the cost may be significantly less than providing biological treatment at the Iona Island plant for the interim period to 2021.

As part of this study, two activities have provided information on how effective in-sewer control activities might be. The first activity included field sampling for solids, soluble and total organics, dissolved oxygen, which was completed at three locations of the Highbury Interceptor and its tributary. Modeling of the sewer system using the hydraulic component of the DHI® Mouse Trap model was carried out to calculate flow velocities at average dry weather flow conditions at key locations along the sewer lines. As well, the Mouse Trap Model water quality components were used to develop mass balances of key parameters such as COD.

The second activity included batch chemically enhanced, primary, and biological treatment and was undertaken in August of 2003 during the dry weather flow period treating both the raw and primary effluent at both IIWWTP and LGWWTP. Standard 96-hour LC_{50} toxicity testing as well as testing for total and soluble BOD, TSS, and surfactants (MBAS methylene blue active substances) was carried out.

The field sampling and testing in the sewer system were limited in nature but did show the following:

- Little trending in soluble organics or TSS occurred in the major trunk sewer from upstream to downstream sections of the Highbury tunnel other than an expected increase in organic and solid loads, consistent with increased inputs along the trunk sewer.
- Flow velocity calculations from the DHI® model indicated that, even during average low flow conditions, the velocities in the main trunks did not decrease to levels where organics and solids would settle out into the invert of the sewers.
- Throughout the trunk sewer system sampled, the dissolved oxygen levels were generally less than 1 mg/L.

From this information, it appears that there are microorganisms at work in the sewers that are utilizing the available dissolved oxygen. But these are not significantly reducing the organic loading.

The six sets of small-scale treatment batch tests (Appendix 5) provided a good indication of organics and surfactants removal that has to be achieved to obtain improved toxicity test results. For these pilot tests, to reduce the frequency of occurrence of acute toxicity, at least 100% chemically enhanced primary or 50% biological treatment (100% of load receiving primary settling plus 50% of ADWF biological treatment) has to be carried out. The required extent of soluble organics removal to improve the LC_{50} test results appears to be 52 to 77%. This is a significant reduction in organics.

The following conclusions can be made:

- Controlling toxicity by reducing the industrial organics load would not be feasible because at the Iona Island plant, the total industrial load only represents about 15-20% of the total BOD load to the plant.
- Achieving a mass reduction in soluble BOD in the range of 50 to 77% would mean converting a portion of the sewer system into a biological or chemical treatment facility. The addition of chemical oxidants such as hydrogen peroxide, potassium permanganate, or ferric salts could not achieve that level of organic destruction at a reasonable operating cost. Creating an in-sewer, tubular reactor biological treatment system would require the equivalent, or greater, capital cost than partial biological (e.g. 50% ADWF) treatment at the Iona Island plant. We are not aware of a major application of these techniques in North America. Transport of the biological solids generated by such an in-sewer system would also be problematic.
- Addition of a chemical agent such as chlorine to lower the level of viable microorganisms to the point where primary effluent contains such low levels of aerobic organisms could be successful at high chlorine doses and retention times. However, environmental risk associated with the formation of chlorinated organic compound would need to be investigated.

- In-sewer treatment is not a feasible option to reduce primary effluent toxicity and until full secondary treatment is implemented at Iona Island WWTP interim upgrades at the plant are required in order to improve toxicity test results.

3.3 SMALL SCALE TESTING

At the Iona Island WWTP, low dissolved oxygen has been identified as the main cause of fish bioassay failures. The low dissolved oxygen has been attributed to high oxygen demand in the plant effluent samples caused by an active population of viable microbes present in the plant influent, combined with high concentrations of readily-degradable organic material (i.e. BOD) in the primary treated effluent.

The small-scale testing program was designed to conduct parallel tests on samples of settled sewage leaving the primary settling tanks. The purpose of the parallel tests was to compare the effectiveness of chemically enhanced primary treatment (CEP) with that of partial biological treatment, and also with that of CEP followed by partial biological treatment, in reducing the acute toxicity of the effluent at Iona Island (acute toxicity as measured by the 96-hour LC_{50} rainbow trout bioassay). An additional batch test was included, to assess the effectiveness of chlorination/dechlorination in improving the chance of passing the 96-hour LC_{50}, by reducing the population of viable bacteria in the plant effluent sample and consequently reducing the initial oxygen demand during the bioassay. Evaluation of partial biological treatment was undertaken using biological waste sludge taken from the Annacis Island WWTP. Each batch test was done in parallel onsite using settled sewage from the IIWWTP, combined with waste biological sludge from Annacis.

Comparisons among the various treatments should be taken as subjective; that is, since parallel tests were conducted on the same sample of settled sewage each time, relative comparisons regarding the effectiveness of one treatment compared to the others are valid. However, the results should not be projected to full-scale WWTP performance. The results of the acute toxicity bioassay testing at Iona Island (96 hr LC_{50}) are summarized In Table 3.2.

TABLE 3.2
IIWWTP TOXICITY RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Pass
CEP	Fail	Pass	Pass	Pass	Fail	Pass
25% Biological	Pass	Fail	Fail	Fail	Fail	Pass
50% Biological	Pass	Pass	Pass	Fail	Fail	Pass
CEP+25% Biological	Pass	Pass	Pass	Fail	Fail	Pass
Disinfected	Fail	Fail	Fail	N/A	N/A	N/A

Twenty five percent (25%) biological treatment was relatively ineffective in improving removals of TSS, TBOD, and SBOD at Iona Island compared to the other treatment processes, and was similarly ineffective in reducing the frequency of acute toxicity in the effluent.

The samples of primary effluent from the Iona Island WWTP contained material that exerted a high oxygen demand in five of the six batch tests. Oxygen starvation was the most probable cause of the observed 100% lethality within the first hour in the control samples in these five tests. Disinfection of the primary effluent at Iona Island was effective in reducing the initial oxygen demand in the bioassay test, but this did not improve the bioassay results. This indicates that reducing the initial oxygen demand by disinfection under the conditions used in this study (i.e. maintaining a total chlorine residual of 2 mg/L for one hour) will probably not improve toxicity testing results.

Chemically enhanced primary (CEP), 50% biological treatment, and CEP followed by 25% biological treatment all showed a 60% improvement in the frequency of toxicity failure compared to the control tests. These three processes appear to be approximately equivalent for use as interim improvements at Iona Island from the standpoint of toxicity reduction and removals of TSS and BOD. None of these processes will produce an effluent that is consistently non-toxic according to the 96 hour $LC_{50,}$ but all can be expected to effect substantial improvements over primary treatment alone.

From the standpoint of effluent quality, chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment should all be considered for interim upgrades at the Iona Island WWTP.

3.4 PLANT CONDITION

The Iona Island plant was originally built in 1962 with upgrades and/or expansion in 1972, 1978, 1983 and 1986. The outfall was upgraded in 1987. The objective of this task was to evaluate the general condition of the various unit processes, tanks and major equipment in order to determine if a component should be replaced in order to integrate the existing primary plant with the proposed secondary plant or to ensure that the treatment facility can be operated satisfactorily for the next 50 years. The purpose of the evaluation of plant condition was not to duplicate the existing operating and maintenance schedule of the plants. Therefore the upgrades already proposed in the 10-year capital plan are not considered in this report.

The plant conditions and options for proposed upgrades can be summarized as follows:

- Influent screens and siphon discharge

  High velocities from the incoming siphons are causing excess stress on the bar screens, forcing screenable materials through bars and forcing grit into crevices. Modification of siphon discharge to reduce velocities, noise and related issues would be difficult and expensive. One option would be to have the existing screens modified to trash racks (25 to 50 or 75 mm spacing) to protect the pumps and to have fine screens located after pumps. One possible location is at the end of the longitudinal grit channel. The location of the fine screens needs further study in relation to retention of the longitudinal grit tanks and distance between the screens from the pump discharge. An allowance of $5 million has been included in the capital cost estimates for the installation of fine screens.

- Influent Pumps

  The six influent pump casings are 40 years old and consideration should be given to replacing those in order to extend the life of the station. This work is considered to be part of on-going maintenance and has not been included in the capital cost estimates.

- Longitudinal Grit tanks

  The concrete wall surface of the longitudinal grit channel needs to be coated in order to correct the problem of exposed aggregate. This work is considered to be part of on-going maintenance and has not been included in the capital cost estimates.

- Grit Removal

  The upgrades to grit tanks and handling are addressed in the August 2002 Dayton & Knight study. The study's conclusions appear sound based on preliminary review of document. Total estimated costs for recommended upgrades (including pre-aeration tanks, influent channel) amount to over $10 million. GVRD has $2.0 million budgeted for 2007 grit system upgrade as part of

10 year capital budget plan. It is assumed that all the recommended upgrades to the grit removal system will be carried out as on-going maintenance and the cost of this work is not included in the capital cost estimates for the plant.

- Flow Splitting to Primary Clarifiers

  Three options have been identified to resolve the problem of uneven flow distribution to the fifteen primary clarifiers.

  1. Hydraulic analysis to size the openings between the influent channel and the pre-aeration tanks. This is least costly option.

  2. Flow splitting chambers, with the first flow splitting chamber having four overflow weirs, followed by four flow splitting chambers with three or four overflow weirs. This option would require more head than available. In conjunction with the installation of new pumps a new grit removal system would be needed. New fine screens could be installed following the grit removal. This options essentially involves new headworks for the plant at an estimate of $50 to $60 million.

  3. Effluent control and submerged launders. This option also offers the advantage of minimizing solids entrainment into the proposed trickling filters or biological aerated filters. The estimated cost for this option is $15-20 million.

- Anaerobic Digestion

  In conjunction with upgrading the plant to secondary treatment in 2021, it is proposed to convert the existing anaerobic digesters from mesophilic to thermophilic mode. Converting the mesophilic digesters to a thermophilic system will require significantly higher heating system components than a mesophilic system. The upgrade should include a heat recovery component to reclaim heat from the 55ºC sludge leaving the digester and raise the temperature of the cold raw sludge. The sum of $2.15 million has been included in the capital cost estimates for this work.

- Structural Repairs

  Structural repairs to deal with seismic issues are described in Section 3.6. Most of the treatment plant building, tank and pipe gallery appear to be in good condition. Minor repairs are required and it is assumed that these will be carried out as part of on-going maintenance.

- Electrical and Control

  The electrical systems of the IIWWTP are very well maintained, and a considerable amount of upgrading has been done recently. More upgrading, both to power and control system is planned over the next 3 to 5 years.

- Data, Communication and Alarms

  The telephone system is old and very basic. Replacement is included in the long-range plan. It may be addressed in the next 1 to 2 years.

3.5 GEOTECHNICAL CONDITIONS

Subsoils at the IIWWTP site consist of deltaic deposits from the Fraser River, comprising unconsolidated silts, sands and silty clays, more than 100 metres in thickness, overlying dense to very dense pleistocene glacial soils. The site has been raised using approximately 4.5 m thick river sand fill prior to construction of the existing structures.

The IIWWTP site has been preloaded in several phases prior to construction of the existing facilities. Major portions of the site have been preloaded prior to the original construction over a 2-year period from March 1959 to May 1961. It is understood that preloads with a 2 to 6 month duration were used for the construction of the various additions to the earlier structures. A review of the preload and settlement history indicates that with an 8.5 m high preload, maximum settlement of 1.82 m was observed over a 2-year duration. Post construction settlement as high as 0.7 m was measured over 35 years.

Preliminary recommendation is a preloading for any additional structure constructed at the site. It also recommended that a setback of 15 m from the nearest edge of an existing structure to the toe of the preload be considered.

It is understood that for the seismic upgrading of the existing structures recommendations given in the National Building Code of Canada (NBCC) 1995 (475 year return period earthquake motion) are to be used. Significantly thick zones of loose sands below the surficial fill zone are expected to liquefy due to the 475-year return period earthquake motion. Liquefaction would likely cause deformation of the ground, dykes, building foundations and floatation of lightly loaded in-ground tanks.

The potential rise in sea level varies between 2 and 9 mm per year. This would result in a sea level rise of about 450 mm over a 50-year period. The existing site grade elevation is approximately 4 m geodetic at IIWWTP. The Sea Island dyke for YVR is 3.5 m geodetic and this elevation provides for the 200-year return plus 0.6 m of freeboard. An additional 0.3 m of freeboard has been recommended for the dyke around Sea Island to account for a possible rise in mean sea level.

The cost of fill and preloading has been included in the capital cost estimates for interim upgrade and build-out to secondary options.

3.6 SEISMIC CONSIDERATIONS

3.6.1 Soil Liquefaction

Liquefaction of subsoil may cause instability and possible failure of the foreshore slopes around the IIWWTP. This may cause distress and possible damage to the various structures at this site. Some form of ground improvement along the waterfront may be required to prevent ground and slope failure.

- It is recommended that a 15 m wide area be densified to 13 to 14 m depth around existing the IIWWTP facility. This densified berm would wrap around the entire facility. The purpose of this densified berm is to minimize the amount of liquefaction induced lateral movement of the ground. Note that liquefaction would still occur inside the non-densified area and below the existing structures. Also, floatation of in-ground tanks may occur. To prevent settlement of buildings and floatation of tanks, other forms of remediation such as soil anchors/mini-piles can be considered.

For any new addition it is recommended that the footprint plus a 5 to 10 m wide envelope around the perimeter be densified. The cost of ground densification around the existing plant has been included in the cost estimates for build-out to secondary. The cost of ground densification under the proposed structures has been included in the capital cost estimates for both interim upgrades and build-out to secondary.

Soil anchors or steel pipe piles can be considered for providing resistance against uplift of buildings and tanks. Soil anchors can also be designed as mini-piles to provide additional axial compression capacity. The anchors can be installed within or around the perimeter of the building, provided that enough headroom for the machinery is available. The cost of soil anchors for the existing structures has been included in the capital cost estimates for the build-out to secondary.

The cost of these measures have been included in the capital cost estimates for the interim upgrades and build-out to secondary as follows:

- Ground densification around existing plant included in cost estimate for build-out to secondary: $1,680,000
- Ground densification around new plant included in cost estimates for interim upgrade ($1,200,000) and build-out to secondary ($1,200,000)
- Soils anchors in existing plant included in cost estimate for build-out to secondary: $8,800,000

3.6.2 Seismic Analysis

The Iona Island Wastewater Treatment Plant is located in one of the highest risk earthquake zones in Canada and all structures in the plant have a high probability of experiencing strong earthquake shaking. Many of the structures in the plant were designed and built before 1980. Before 1980 satisfactory earthquake assessments were not required by code with respect to locations (such as Richmond). Also prior to 1980, code requirements to design water- retaining structures for earthquake conditions were less stringent than the current National Building Code of 1995 and the British Columbia Building Code of 1998. The National Building Code of Canada was revised in 1985 and this Code introduced new earthquake design requirements, standards and adequately accounted for seismic forces. Prior to 1985, liquefaction was not typically considered when carrying out geotechnical assessments. As indicated above, liquefaction could occur with possible vertical movement of up to 250 mm (half of this could be treated as differential over a 5 m distance) and horizontal movement of up to 300 mm. Such movement could cause heavy damage to structures.

An overview and seismic assessment of the structures at the Iona Island WWTP was carried out. It involved a review of the existing drawings of the structures and applying the current National Building Code of Canada to analyze the structures for a 1 in 475 year return period design basis earthquake. A summary of the assessment of the Iona Island WWTP structures for the six stages of construction is summarized as follows:

- Digesters 1 and 2 will suffer almost non-repairable damage and leaks from the design earthquake.

- Precast panels anchored to Digesters 1, 2, 3 and 4 require some supplementary bracing to prevent their collapse.

- The roofs of Stage II Preaeration and Sedimentation Tanks require upgrading.

- Otherwise, the rest of the structures are generally adequate to accommodate seismic forces from the design earthquake.

- Some of the structures in the plant could suffer various types of damage due to uneven ground movements, as a result of liquefaction. Ground surfaces will crack and cracks will run beneath structures. Unless liquefaction mitigation measures are implemented, this could cause damage to structures, especially at the expansion joints between tanks.

- Waterlines, gas lines and sewer conduits in the plant area may suffer some damage and leakage due to differential settlement. Particularly the joints of pipes and outfall conduits, resting on a dyke, may suffer damage due to liquefaction movements.

The cost of the above improvements are detailed in Appendix 8 and are included in the capital cost estimates for the build-out to secondary.

4.0 SELECTION OF PREFERRED TREATMENT OPTION – BUILD OUT TO SECONDARY

4.1 SUMMARY OF ALTERNATIVES CONSIDERED

An extensive review of secondary treatment processes was undertaken in order to evaluate the most effective and affordable options for the IIWWTP and LGWWTP. These options included various configurations involving fixed-film biological treatment, suspended growth treatment, anaerobic treatment and several feasible combinations of these configurations. The following section briefly summarizes the various scenarios. Complete descriptions are discussed in Appendix 4.

4.1.1 Fixed Film Treatment

- Trickling filters (TF) utilize a specially designed media on which biofilms develop when exposed to the primary effluent. Primary effluent is distributed uniformly around the surface of the media, allowing the water to ”trickle” down over the biofilm. Several types of trickling filters were reviewed, including standard rate and high rate (or roughing trickling filter). Standard rate TFs are designed to achieve greater than 90% BOD_5 and TSS removals. Rough trickling filters are only designed to achieve 40 ~ 70% BOD_5 and 70 ~ 80% TSS removal. Historically, TFs were avoided in cold climates due to excessive temperature loss in the wastewater; however proper engineering can limit overall temperature losses to less than 1.5 °C.

- The trickling filter/ solids contact (TF/SC) process utilizes a short HRT aerobic suspended growth chamber following the TF. This allows the biomass that has sloughed off the TF to be concentrated and aerated, improving the sludge settling characteristics in the clarifiers. In addition further removal of BOD can occur. Variations on the TF/SC involve the addition of a re-aeration tank prior to sludge recycling. TF/SC applications are common in North America and within the GVRD; they are a proven technology and can achieve a high quality effluent.

- A rotating biological contactor (RBC) involves a fixed film biomass, which grows on the surface of closely spaced circular disks. These disks are mounted around a central horizontal shaft around which the disks rotate though the effluent. Aeration is achieved by placing the disks so that the top 60% is exposed to the air; the portions of the biomass exposed to the effluent achieves BOD reduction, while the portion above the effluent is aerated. The shear stress created by rotating the disks through the effluent allows dead biomass to shear off. Secondary clarifiers are used to remove TSS and excess biosolids. RBCs have been commonly applied in the wastewater field and can be designed to achieve a high level of effluent quality.

- Biologically aerated filters (BAF) were evaluated as one of the interim options. A summary description of this process is included in Section 5.1.

4.1.2 Suspended Growth

- Conventional activated sludge (CAS) and high rate activated sludge (HRAS) were evaluated as interim options. A summary description of this process is included in Section 5.1.

- Oxidation ditches are a variant of the extended aeration activated sludge process. A long retention time is used to stabilize the primary effluent and reduce the solids load. Oxidation ditches are very stable and resistant to shock loading due the high hydraulic retention time (HRT), between 12 and 36 hours. The long HRT also results in a reduction in the production of biosolids. Compared to the other secondary treatment technologies oxidation ditches require a large footprint. A portion of the excessive solids and biomass separated in the secondary clarifiers is recycled back to the oxidation ditch to maintain the biological culture.

- High purity oxygen activated sludge uses pure oxygen (>95%) rather than air for aeration in the activated sludge basin. The purity increases the amount of oxygen available for oxidation/reduction reactions by the microorganisms, hence allowing a higher MLSS concentration and smaller aeration tank size. Additional land is required to provide oxygen production facilities and a slightly large clarifier is required compared to CAS. However, overall footprints are generally lower than CAS plants.

- Multi anoxic step feed utilizes alternating anoxic/aerobic tanks to achieve carbonaceous BOD and nitrogen removals. Primary effluent is introduced in each of the anoxic zones to maximize the BOD removal associated with denitrification. Reduced aeration is required in this option versus the CAS due to the “denitrification credit” from the anoxic zone. However it is a more complex process to monitor and operate.

- Pre-anoxic activated sludge is a variation of the CAS, such that a high level of nitrogen removal is achieved. The aeration tank is preceded by an anoxic tank which reduces some BOD during denitrification. This results in a reduced aeration tank size and air requirements compared to CAS.

- Sequencing batch reactors (SBR) combine the activated sludge, nitrification/denitrification (if included), and clarification into a single reaction tank. The tank is filled with primary effluent, aerated to provide BOD removal, settled and then decanted. SBRs utilize a batch process and require multiple tanks in parallel to achieve continuous operation. Control systems for SBRs are typically more complex than CAS, however, SBRs are more resistant to shock loads.

- Membrane activated sludge (MAS) is a modification of the CAS which combines the clarification process and the aeration process into one tank. A membrane (0.1 micron to 0.4 micron pore size) is used to separate the treated effluent from the MLSS. Membrane performance allows for the MLSS concentration to be increased to around 10,000 mg/L, resulting in a reduction in the aeration tank size. Fouling and bulking problems commonly associated with clarifiers are avoided, however membrane fouling can limit overall system performance. MAS is a relatively new process and few installations exist in North America.

- Deep shaft technology (Vertreat®) is a proprietary process which utilizes a vertical shaft 120 to 150 metres deep. Primary effluent is forced into the tank and aerated with a compress air. The increased pressure at the bottom of the tank increases oxygen solubility in the water, resulting in a supersaturated solution. The treated effluent is extracted from the bottom of the tank and clarified in a separated floatation unit. In the floatation unit the air present in the supersaturated effluent becomes gaseous and floats the biological solids to the liquid surface. The thickened activated sludge is either returned to the reaction column or wasted.

- The upflow sludge blanket filtration clarifier (USBF) combines anoxic and aerobic treatment zones into a single clarification process. The primary effluent enters the USBF's anoxic zone where BOD reduction is achieved during denitrification. Denitrified effluent flows into an aerobic zone where further BOD removal is achieved along with nitrification. Finally, the effluent flows into a central clarification zone where the solids are separated and a portion returned to the anoxic zone as return activated sludge.

4.1.3 Anaerobic Processes

Anaerobic processes are common treatment techniques for wastes containing elevated BOD concentrations. Overall performance is reduced with more dilute wastes, such as domestic sewage, however there are applications in which this waste is treated anaerobically. Effluent performance for anaerobic processes is typically lower than aerobic techniques due to a reduction in solids settleability. Ammonification and cell lysis are common problems with anaerobic processes making them unattractive for the build-out to secondary processes.

- Continuously stirred tank reactor (CSTR) low rate bioreactor is an anaerobic process in which the primary effluent enters an enclosed completely mixed tank in the presence of a biosolids concentration. The BOD is converted to methane, carbon dioxide, water and hydrogen. The low rate bioreactor requires a separate clarifier for solids separation; no sludge recycling is employed for dilute wastes. This process is resistant to shock loads and requires low energy. Odours and corrosion are common problems associated with anaerobic processes.

- Upflow anaerobic sludge blanket (UASB) bioreactor is the most common high rate anaerobic process in North America. The waste is introduced at the bottom of the tank. The biological solids develop into granules, increasing the settleability. As the effluent flows up through the tank BOD is converted to methane, carbon dioxide, water and hydrogen and the solids are separated in a sludge blanket. Solids are removed from the bioreactor, while gas, liquid and excessive solids are removed in a separator at the top of the tank. This process can handle high organic loading, however, biomass is subject to washout and offensive odours are generated.

- Packed bed filters and fluidized bed reactors are anaerobic processes in which a biofilm develops on a specialized media. Waste is introduced at the bottom of the tank where it flows vertically through a biological sludge blanket where BOD is converted to methane, hydrogen, water and carbon dioxide. Flow rates are low enough to prevent fluidization of the bed, whereas in a fluidized bed reactor the influent flow is sufficiently high to result in the suspension of the media. These reactors are capable of handling high BOD loading conditions, however plugging of the media in a packed bed filter due to excessive biological growth can occur, while non-uniform biological growth may occur in the fluidized bed reactor.

- The bulk volume fermenter (BVF®) is a proprietary process marketed by ADI. Effluent is pumped into the bottom of an earthen/concrete tank; as the flow rises in the tank it passes through an anaerobic sludge blanket during which BOD is converted to methane, carbon dioxide, hydrogen and water. Sludge is either wasted or recycled from the bottom of the tank to maintain the biological culture. Biogas is collected from a floating geomembrane at the top of the tank. An HRT of 7 hours results in a large land area requirement. However, as a result the system is relatively resistant to shock loads.

- Hybrid system which combines the above process, such as UASB and fixed film, were reviewed. In the hybrid systems, the bottom of the tank consists of a sludge blanket while the upper portion utilizes a fixed film media. This process is efficient for treating waste with a high COD and low TSS concentration. Plugging of the media from suspended biomass can result in reduced performance in the fixed film portion. Biogas is collected for reuse or flaming.

4.1.4 Fixed Film Suspended Growth

Trickling filters/activate sludge (TF/AS) combines two of the previously described aerobic options. Effluent quality in the two-stage process is generally better than could be achieved through the operation of either process alone.

Kaldnes® Moving Bed™, Captor® and Linpor® processes are proprietary technologies that utilize small, neutrally buoyant media which are suspended in an aeration tank. The high surface area of the media permits a high concentration of biomass resulting in a smaller tank size than CAS. Sloughed biofilm is removed from the treated effluent in a separate clarification process.

4.1.5 Submerged Attached Growth and Miscellaneous Option

A number of proprietary submerged processes, including: Ringlace®, BioMatrix® processes and Bio-2-Sludge® process were evaluated. These processes utilize proprietary media submerged in an aeration basin of which biofilms develop. These processes have a limited track record and can be subject to microbiological problems.

Advanced oxidation is a process which utilizes the high oxidizing potential of the hydroxyl radical (OH·) to convert BOD into smaller, easily degradable organic constituents. The hydroxyl radical is formed from the reaction of ozone with UV light or ozone and hydrogen peroxide. The feasibility of this process and performance are highly dependant on the wastewater characteristics, hence pilot testing is always required prior to any assessment.

4.1.6 Chemically Enhanced Primary and Biological Treatment

Several configurations of chemically enhanced primary (CEP) and biological treatment were discussed as part of the interim options; a summary description of this process in included in section 5.1.

4.2 RESULTS OF FIRST LEVEL OF SCREENING

An initial screening process was utilized to determine the most feasible options for the build-out to secondary process. A complete description of the method is presented in the section 8 of Appendix 3. A summary is included in section 5.2 of this report. The initial screening resulted in the following options being selected for the build-out to secondary process:

- Option 1: Primary treatment followed by 2 x ADWF TF/SC,
- Option 2A: Primary treatment followed by 2 x ADWF CAS,
- Option 2B: Primary treatment followed by 2 x ADWF including the LGWWTP flow,
- Option 3. CEP treatment followed by 60% of 2 x ADWF CAS.

Each of these build-out to secondary options allow for an easy transition from the interim processes selected for detailed assessment. These options were then assessed in more detail by developing preliminary sizing, costs and effluent performance based on typical design values as summarized in the next section.

4.3 SUMMARY OF PROCESS OPTIONS THAT PASSED FIRST LEVEL OF SCREENING

Typical design values were used to undertake a preliminary design for each option; these values were used in order to provide a consistent level of comparison for all options. Schematic process flow diagrams, conceptual plant layouts, footprint requirements, sludge production, effluent quality projections, capital and O&M cost estimates, process flexibility and other factors were developed and evaluated. Each option is expected to achieve an average effluent quality of 20/20 mg/L BOD/TSS. Sludge projections for the recommended option are presented in Section 6 of this report. The following is a brief summary of options that passed the first level of screening.

4.3.1 Option 1 – Primary + 2 x ADWF TF/SC

The preliminary and primary treatment units are designed to treat the flow collected from the VSA. Six trickling filters and three solids contact tanks are designed to treat 2 x ADWF at 100% MM loading (1,000 MLD and 81 t/d BOD and 57 t/d TSS, respectively); excess flow is discharged directly to the effluent pump station. Sixteen secondary clarifiers are used to remove excess biological solids and TSS from the TF/SC plant. Primary and secondary sludge will be thickened using the existing gravity and three additional DAF thickeners, respectively. Thickened sludge will be stabilized through mesophilic or thermophilic anaerobic digestion (depending on market demand for biosolids). The estimated footprint is 63,000 m^2 with an estimated capital cost of $346,020,000.

4.3.2 Option 2A – Primary + 2 x ADWF CAS

This option is similar to option 1, however sixteen CAS tanks are used in place of the TF/SC process. Units are sized to treat 100% MM BOD and TSS loading (81 t/d and 57 t/d, respectively) at a maximum flow of 2 x ADWF (1,000 MLD). Solids treatment and reject water will be treated using the same approach as option 1. The total footprint is estimated as 92,700 m^2 with a capital cost of $396,522,000.

4.3.3 Option 2B – Primary + 2 x ADWF CAS including the LGWWTP Flow

This option is an extension of option 2A which will treat the flow from the VSA and the NSSA (current LGWWTP flow). Nineteen CAS tanks are required to treat the 1,232 MLD flow at a loading of 98 t/d BOD and 72 t/d TSS. A total of 19 secondary clarifiers, one additional gravity thickener, 4 DAF thickeners and 3 digesters are required to treat the flow. A total footprint of 112,300 m^2 is required at an estimated capital cost of $446,660,000.

4.3.4 Option 3 – CEP + 60% of 2 x ADWF CAS

CEP treatment is used to reduce solids and organic loadings on the secondary treatment facilities. An additional 20% BOD and 30% TSS removal is expected using CEP. Eight CAS tanks are required to treat the maximum flow of 60% of 2 x ADWF (600 MLD) with a maximum design load of 34 t/d BOD and 14 t/d TSS. Ten (10) secondary clarifiers, 2 gravity thickeners, 1 DAF and 2 additional digesters are required. The total footprint is 57,000 m^2 with an estimated capital cost of $247,997,000.

4.4 RESULTS OF SECOND LEVEL OF SCREENING

A second level of screening was undertaken to select the final recommended option for the IIWWTP. Descriptions of the procedure for this screening process are presented in Section 10 of Appendix 3 and summarized below.

The following factors were considered for the selection of the short list of preferred process options. In addition to cost, environmental and social related issues, there are several factors of technical nature that need to be considered for process selection.

1. <u>Cost and Technical</u>

1.1 <u>Cost Factors</u>

- <u>Capital cost</u> – Total capital cost including construction, engineering and contingency.
- <u>Operating and maintenance cost</u> – Operating cost of existing primary plant plus additional cost for the upgrade.
- <u>Lifecycle cost</u> – Lifecycle cost include both capital cost and operating and maintenance cost and is a measure of all future cost expressed in today's dollars.

1.2 <u>Technical</u>

- <u>Footprint of plant</u> – What portion of the site will be utilized by each process option and will the plant fit on the available space.
- <u>Ability to expand on site</u> – What portion of the site will be available for future expansion using the same process.
- <u>Ease of phasing</u> – This factor is related to the ease of building the plant in phases.
- <u>Ability to upgrade for nitrogen removal</u> – Can the process be upgraded for possible future requirement to remove nitrogen.
- <u>Resiliency of process</u> – Is the process stable in case of shock loading from sudden variations in flows and loads.
- <u>Compatibility of process with other GVRD plants</u> – Is the staff from other GVRD plants familiar with the process and how easily could staff from other plants be trained.

2. <u>Environmental</u>

- <u>Energy use</u> – Includes the energy used by both the liquid and solids stream processes.
- <u>Production of greenhouse gases</u> – Production of methane and carbon dioxide resulting mainly from the digestion of sludge produced.

- Sludge production – Annual volume of sludge produced.
- Effluent quality – Effluent quality for BOD_5 and TSS.
- Impact on wildlife habitat – This factor applies to Iona Island site only which has tidal wetland adjacent to the existing primary plant.
- Production of aerosols – Production of small droplets which can be carried off site by wind.

3. Social

- Visual impact – Size of above ground tankage and structures
- Risk of odours if odour control system fails – Risk refers to the comparable generation of odours of the processes and their dependence on odour control.
- Traffic generation – This is mainly tied to truck traffic for hauling of biosolids off site for recycling and reuse.
- Land tenure concerns and issues – This factor applies only to Lions Gate plant only which is located on leased lands.

The treatment process options that passed the first level of screening were evaluated against all the above factors. A sensitivity analysis was carried out by using three different weightings against the three above categories.

- Cost and technical @ 50%; Environmental @ 25%; Social @ 25%
- Cost and technical @ 30%; Environmental @ 50%; Social @ 20%
- Cost and technical @ 30%; Environmental @ 20%; Social @ 50%

As discussed in Section 5, 6, 9, and 10 of this summary report, the process options for interim upgrades and for build-out to secondary were evaluated using these factors and to select the short list of preferred process options.

Based on the analysis summarized in Table 4.1, the recommended option for build-out to secondary at the IIWWTP is:

- Option 1 - Primary treatment plus 2 x ADWF TF/SC.

The concurrent assessment for the build-out to secondary treatment at the LGWWTP indicated that BAF was a more viable option for secondary treatment than originally assessed in the first level of screening, due to lower capital costs, smaller footprint, and the potential for increased application at larger facilities. For these reasons, BAF was added as a viable option for the IIWWTP build-out to secondary. This is discussed further in Section 4.5.

TABLE 4.1
IONA ISLAND BUILD-OUT TO SECONDARY TREATMENT

	Option 1 TF/SC		Option 2 CAS		Option 3 CEP + 60% CAS	
	Points	Rank	Points	Rank	Points	Rank
Cost & Technical @ 50%	89.6	1	85.0	2	84.7	3
Environmental @ 50%	91.6	1	82.8	3	82.9	2
Social @ 50%	85.5	2	88.4	1	84.2	3
Overall Rank	1		2		3	

4.5 PREFERRED PROCESS OPTIONS FOR BUILD-OUT TO SECONDARY

As discussed above, there are two preferred process options for the build-out to secondary at Iona Island: trickling filter/solids contact (TF/SC) and biological aerated filter (BAF).

4.5.1 <u>TF/SC</u>

The TF/SC process is recommended as the preferred option for the build-out to secondary at the IIWWTP based on the following reasons:

- Lowest life cycle cost,
- Lower operating and maintenance costs than CAS and HRAS,
- Compatibility with other GVRD WWTPs,
- Smaller footprint than CAS,
- Ease of phasing from the interim option to the build-out option, and
- Lower energy consumption costs.

The required units for TF/SC process were designed based on the maximum month loads (83 t/d BOD and 60 t/d TSS) and a maximum flow of 2 x ADWF (912 MLD). It should be noted that the <u>upper envelope</u> MM loads and ADWF values were used for the design during the screening process, whereas during the preliminary design the <u>design case</u> flow and loads were used. Table 4.1 summarizes the number of units and required sizing for the TF/SC process.

TABLE 4.2
SUMMARY OF PROPOSED UNIT PROCESS AT IIWWTP (BUILD OUT TO SECONDARY)

Trickling filters	6 @ 44 m diameter, 6 m high
Solids contact	4 @ 78 m long, 18 m wide
Secondary clarifier	16 @ 41 m diameter, 4.5 m SWD
DAF	3 @ 21.5 m diameter, 3.5 m depth
Digesters	4 @ 32 m diameter, 10.6 m depth
Centrifuges	4 @ 145 m^3/hr

4.5.2 BAF in Place of TF/SC

During the analysis of build-out to secondary treatment options for the LGWWTP the biological aerated filter (BAF) option was determined to be less costly than estimated in the initial process screenings (similar to TF/SC option). In addition, the required footprint for the BAF was significantly smaller than the TF/SC process (45 % of the size). This led the evaluation team to include BAF as a viable option for the IIWWTP.

In summary, BAF was added to the short list of options for the Iona Island plant for the following reasons:

- The cost of BAF is not as high as anticipated earlier and is comparable to TF/SC.
- BAF has the lowest footprint of all biological processes and this would allow for greater site utilization and expansion.
- A smaller footprint would reduce site preparation (fill and pre-loading) and would reduce the need for hauling a large volume of dewatered sludge that is currently stored on site.
- Concerns had been raised regarding a lack of similar sized WWTP utilizing BAF as would be required for the Iona Island facility. However, wastewater treatment plants using BAF are constantly being developed and this matter should be re-examined at the time of design.

As with TF/SC, the headworks and primary sedimentation tank will treat the entire flow collected from the Vancouver Sewage Area (VSA). The BAF process will provide capacity of two times of the average dry weather flow at 100% of the maximum month loading. The BAF process does not require final clarifiers to remove the TSS and biological sludge generated in the biological process, rather, the solids are removed in the BAF back wash cycles. The flow in excess flow of two times of the ADWF will bypass the secondary treatment units and discharge directly to the outfall pump station after the primary treatment.

The biological aerated filter was sized based on using the Biofor® system as supplied by Degremont®. A layout plan using the Biofor® BAF process was prepared in Figure 4.1 mainly for a comparison of site utilization with the TF/SC process. Sludge production of BAF option is estimated about 20% higher than the TF/SC option.

4.5.3 Layout Of Proposed Secondary Plant

Several options were considered for the layout of the IIWWTP build-out to secondary facilities. The proposed layout minimizes the impact on the existing wetlands located on the south half of the site and avoids the existing grit dump located on the east-central part of the site. To achieve minimum head loss, the liquid process is located east of the primary clarifiers. Primary effluent will flow from the existing channel to a new pipeline connected to the secondary treatment flow split. Flow up to 2 x ADWF will be pumped to the secondary facilities, and excess flow will be discharged to the effluent channel. Treated effluent flows via an open channel to the existing effluent junction box.

The solids handling facilities are located on the west side of the site, in the northeast lagoon. Additional sludge blending and storage tanks have been provided to distribute sludge to the existing and future digesters. By locating the solids facilities on the west side of the existing plant the sludge pumping between existing digesters and thickeners is minimized. The proposed TF/SC layout is presented in Figure 4.2.

FIGURE 4.1

LAYOUT OF BAF AT IIWWTP

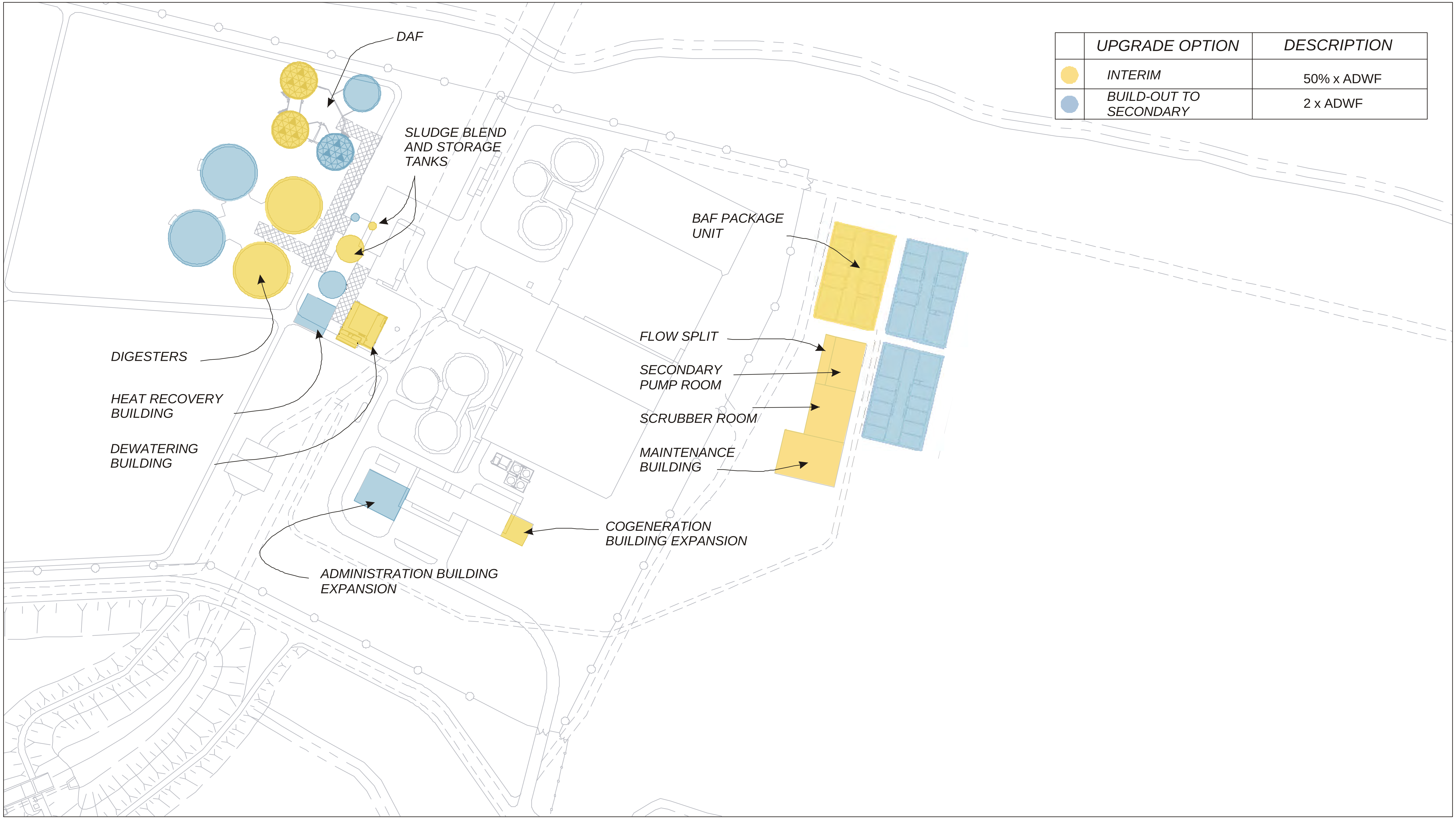
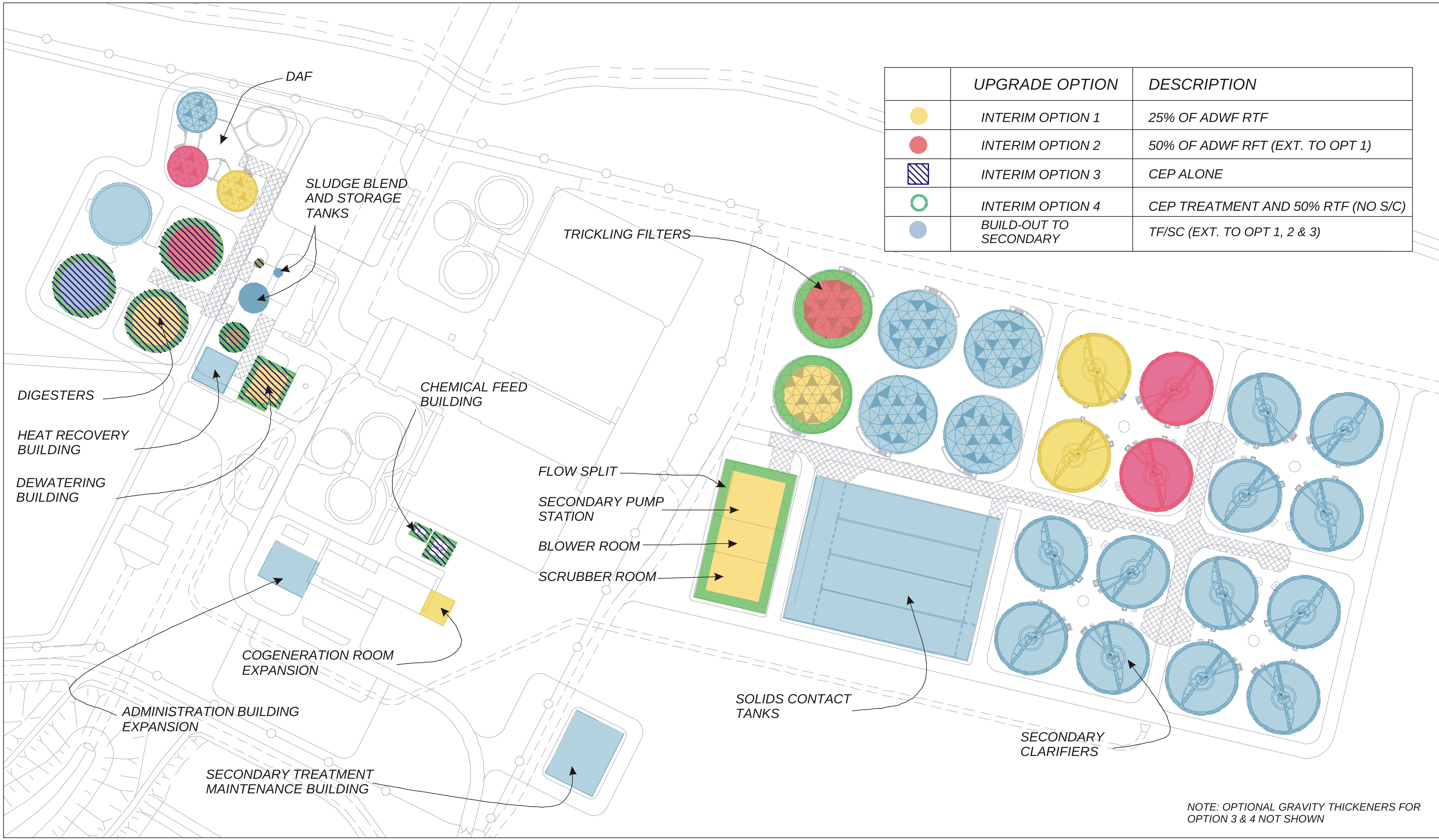


FIGURE 4.2

LAYOUT FOR TF/SC AT IIWWTP



	UPGRADE OPTION	DESCRIPTION
	INTERIM OPTION 1	25% OF ADWF RTF
	INTERIM OPTION 2	50% OF ADWF RFT (EXT. TO OPT 1)
	INTERIM OPTION 3	CEP ALONE
	INTERIM OPTION 4	CEP TREATMENT AND 50% RTF (NO S/C)
	BUILD-OUT TO SECONDARY	TF/SC (EXT. TO OPT 1, 2 & 3)

4.6 SUMMARY OF ESTIMATED COST

Capital cost estimate for the build-out to secondary options are summarized in Table 4.3. These costs have been revised from the original estimates presented in Appendix 4 to account for the change in sludge handling, from complete lagoon treatment to partial mechanical dewatering.

The estimated operating and maintenance cost are summarized in Table 4.4. It is estimated that the build-out to secondary process will require a staff of 80 persons and maintenance costs will be fixed at 0.80% of the capital cost.

Life cycle costs indicated in Table 4.5 were calculated assuming that all the work required for build-out to secondary takes place over a three-year period from 2018 to 2020.

TABLE 4.3
CAPITAL COST ESTIMATES
BUILD-OUT TO SECONDARY AT IIWWTP

YEAR	Build-out to Secondary 2036	
Option	Option 1 TF/SC	Option 2 BAF
CAPITAL COSTS		
Site Improvements	$34,835,000	$21,034,000
Chemical Feed	$0	$0
Biological Aerated Filter	$0	$92,600,000
RTF/TF/SC	$50,370,000	$0
Solids Contact Tank	$10,541,000	$0
Secondary Clarifiers (SCL)	$45,196,800	$0
Gravity Thickeners	$0	$0
Sludge Blending Tank	$1,000,000	$1,000,000
DAF Thickeners	$23,086,800	$25,395,480
Digesters	$32,054,000	$35,259,400
Mechanical Dewatering	$25,800,000	$25,800,000
Site Works	$14,022,000	$7,982,000
Admin/Maint. Building	$4,000,000	$4,000,000
Control System	$6,610,000	$6,190,000
Expansion of Cogeneration	$11,700,000	$11,700,000
Odour Control	$3,000,000	$500,000
Existing Facility Upgrades	$69,150,000	$69,150,000
Sub-Total	$331,366,000	$300,611,000
Division 1 Cost	$7,413,000	$6,989,000
Engineering	$53,018,000	$48,098,000
Project Management/QA/QC	$13,255,000	$12,024,000
Contingency	$99,410,000	$90,183,000
Total Capital Costs	$504,462,000	$457,905,000

TABLE 4.4
OPERATING AND MAINTENANCE COST ESTIMATES
FOR BUILD-OUT TO SECONDARY AT IIWWTP

YEAR	Build-out to Secondary 2036	
Option	Option 1 TF/SC	Option 2 BAF
O&M COSTS		
Labour	$5,365,000	$5,365,000
Chemical Costs	$450,000	$517,000
Residuals Management	$6,938,000	$7,970,000
Energy/Power	$2,458,000	$3,645,000
Repair/Maintenance	$6,812,000	$6,440,000
Administration and others	$1,784,000	$1,757,000
Total (O&M Costs)**	$23,806,000	$25,691,000
Total (O&M Costs)***	$14,714,000	$16,600,000

Notes
**: Entire plant O/M costs including existing primary plant and upgrade
***: Upgrade O/M costs only (existing primary plant excluded)

TABLE 4.5
LIFE CYCLE COST FOR BUILD-OUT TO SECONDARY AT IIWWTP

YEAR	Build-out to Secondary 2036	
Option	Option 1 TF/SC	Option 2 BAF
Discounted Total O&M Cost	$82,217,000	$92,756,000
Discounted Capital Costs	$198,805,000	$180,457,000
Total Capital and O & M Costs at Present Value	$281,022,000	$273,212,000

The life cycle cost (LCC) of the build-out option is included in Table 4.5. The LCC are based on the following parameters:

- Discount rate: 6% p.a.
- Base date for costing: November 2003
- Evaluation period for build-out: 2018 to 2060
- Construction period for build-out: 2018 to 2020 (1/3 of capital each year)

The LCC analysis differs from the capital works program in the timing assumed for expenditures. This difference results from the proposed schedule for the upgrading.

In the LCC analysis, the cost of each option is separate and stand-alone and the cost of interim treatment cannot be added to the cost of build-out. This is because it would be necessary to deduct the capital cost of the interim works from the build-out cost to arrive at a correct result. As indicated above, for the LCC analysis of the interim options it has been assumed that interim upgrades will be constructed by 2010. No operating costs have been included for the build-out options between 2004 and 2020. This is because the costs are dependent on the interim option chosen.

5.0 SELECTION OF PREFERRED TREATMENT OPTIONS – INTERIM TREATMENT

5.1 SUMMARY OF ALTERNATIVES CONSIDERED

A review of thirteen treatment processes was conducted as part of the selection of the preferred options for the interim upgrade for the IIWWTP. The options range from exclusively chemical/physical process to complete biological treatment. Several options involved a combination of chemical, physical and biological treatment. All thirteen options were evaluated and rated prior to receiving a first level of screening. The options that passed the first level of screening where evaluated in more detail before selecting the short list of preferred option. All the options that were considered are briefly reviewed in this section. Detailed descriptions are presented in Appendix 3.

5.1.1 Physical/chemical Processes

- Chemically enhanced primary (CEP) involves the addition of a coagulation chemical, typically alum, iron or calcium generally in conjunction with patented polymers. Coagulation chemicals increase the removal of suspended and colloidal particles and to a lesser extent, dissolved compounds, such as phosphorus. CEP is a very reliable process which is commonly utilized in North America and will show measurable improvements in effluent quality immediately following the process initialization. Additional chemical storage tanks and pumping rooms are required for upgrading to CEP.

- CEP with lamella retrofit involves the addition of lamella plates to the primary clarifiers to improve the clarification efficiency. Lamella plates are closely space plates inclined at 45° or 60° from the horizontal and covering approximately 80% of the sedimentation tanks. An additional 10 to 15% sludge volume would be expected compared to CEP alone. The increased sludge production would require the upgrade of the existing sludge collection/pumping, dewatering and stabilization processes. The capital costs and operation costs will be higher than compared to CEP alone.

- DensaDeg® is a proprietary process combining coagulation, flocculation and lamella plate clarification in a single unit process involving two tanks. In the first tank, the “reactor zone”, primary effluent is mixed with recycled solids from the second tank, the “clarification zone”. The mixed solids are combined with a metal coagulant and polymers to develop a dense floc. The flocs flow from the reactor zone to the clarification zone through an upflow daft tube. The remaining solids are pumped from the bottom of the tank directly to the digesters. The DensaDeg® process requires a small footprint (~2,800 m^2) for the IIWWTP and would require the construction of dedicated tankage, rather than a retrofit of existing facilities.

- Ballasted flocculation (Actiflow® as marketed by John Meunier®/US Filter®) is a high rate clarification process. Primary effluent is mixed with a coagulant to destabilize the particles. Following an initial mixing tank a coagulation polymer is added along with microsand. The microsand acts as a seed for floc formation and raises the particle density to increase the clarification rate. The settled sludge is passed through a hydrocyclone to separate the microsand from the sludge, the microsand is then recycled to the Actiflow® influent. The Actiflow® system would require the replacement of the existing sedimentation tanks, increased labour to clean the lamella plates and the replacement of the microsand. This is a robust process, which can be initiated in 10 to 15 minutes. Common applications can be found in the both the water and wastewater fields.

5.1.2 Partial Biological Treatment

With this option, partial biological treatment processes would treat between 25 and 50% of the dry weather flow and maximum loads. The remaining wastewater would receive only primary treatment. The effluent from the biological plant would be mixed with the primary treated waste prior to the ocean outfall.

- Conventional activated sludge (CAS) has been in use for wastewater treatment for more than 50 years. The primary effluent is aerated in a basin containing an elevated biological solids concentration, about 2,500 mg/L, which allows for biological removal of BOD. Clarifiers are used to separate the solids, which are either wasted or recycled (25-50% recycled rate) back to the aeration basins to maintain the elevated solids concentration and microbiological population. A high level of attention and knowledgeable operators are required to minimize the impact of process upsets, which could reduce effluent quality for several days at a time.

- High rate activated sludge (HRAS) utilizes a similar aeration/clarification process as the CAS. The difference is that HRAS utilizes a higher volumetric BOD loading and a higher F/M ratio. This results in a reduction in the effluent water quality and a decrease in the reliability of the process. HRAS is considered as an option for the interim and could be expanded to a CAS system for the build-out at IIWWTP.

- Trickling filter (TF) are biological treatment processes where the biological growth is supported as a slime layer on the specially designed media. The sewage is “trickled” over the media and then treated in a secondary clarifier to remove excessive solids. TFs designed to treat only a portion of the BOD_5 (such as 50%-75%) are termed roughing trickling filter (RTF).

- Biologically aerated filters (BAF) are submerged reactors, which contain a porous media through which the primary effluent is pumped either upwards or downwards. Fine air bubbles are introduced to the system from the bottom of the filter bed, the air provides the necessary oxygen for the biofilms that develop on the surface of the media. Treated water is stored for use during a backwash cycle to remove the excess biological growth. BAFs require a small footprint and can accommodate varying flow rates through the use of multiple filter units.

5.1.3 CEP with Biological Treatment

- This process combines two of the previously discussed options, chemically enhanced primary treatment and biological treatment, such as RTF. Both options are commonly applied in the wastewater field, however there are few applications where both are used concurrently. This process option will produce an increase in chemical sludge and biological sludge as compared with the existing primary plant. Additional footprint area is required due to the construction of a biological process and additional solids handling facilities are needed to handle the increased sludge volumes.

5.1.4 Dissolved Air Floatation

- Dissolved air floatation (DAF) is a process which utilizes fine air bubbles to float solids to the surface of the tank where the sludge is skimmed off into an adjacent hopper. A DAF system could be added to aid the primary treatment system by treating a portion of the degritted flow. Solids removal achieved in DAF system is typically around 85%. Only BOD_5, which is associated with solids, can be removed by DAF, which will result in a limited effluent toxicity reduction.

5.1.5 Primary Treatment with Add-On Chemicals

Several options were examined which could be used to improve effluent quality through the use of oxidizing agents such as chlorine, ozone and hydrogen peroxide.

- Chlorination utilizes the oxidizing power of chlorine to kill microorganisms, which will consume oxygen in the wastewater in the presence a substrate. During oxidation a portion of the BOD_5 will be removed. A dechlorination system, such as sulphur dioxide, is necessary prior to the ocean outfall.

- Ozonation is a powerful oxidant, which can reduce BOD_5 and microorganisms in the wastewater. Ozone is very unstable and is generated on site, which eliminates the risks of chemical transport and storage. A separate ozonation tank is required to house the diffuser equipment necessary to introduce the ozone to the wastewater. As with chlorination no improvement in TSS removal can be achieved, but a limited reduction in toxicity will be achieved due to a decrease in the BOD_5.

- Hydrogen peroxide is a third commonly applied oxidant, which will have similar impacts as chlorine and ozone. In water hydrogen peroxide will be oxidized to oxygen gas and water. This process may potentially remove a very small portion of suspended solids through floatation, similar to the DAF process. Improvements in BOD_5 levels and a reduction in toxicity can be anticipated.

5.2 RESULTS OF FIRST LEVEL OF SCREENING

The first level of screening involved an initial pass/fail analysis for each of the options based on the following criteria:

- Proven technology,
- Discharge requirements,
- Reliability,
- Site suitability.

Options that did not pass each criteria were assessed to determine if the results would change due to technological improvements, if so they were carried over to the evaluation stage. Options which passed the initial screening were evaluated using a "Delphi" ranking exercise. The objective of the Delphi method is the production of suitable information for decision making. The Delphi method is based on a structured process for collecting and distilling knowledge from a group of expert by means of questionnaires interspersed with controlled opinion feedback. The technique allows experts to deal systematically with a complex number of options. The essence of the technique is to send a series of questionnaire by email to a pre-selected group of expert. These questionnaires are designed to elicit and develop individual responses to the questions posed and to enable the experts to refine their views as the groups' work progresses. The main point behind the Delphi method is to overcome the disadvantage of conventional committee actions. The "Delphi" exercise allowed a panel of experts to rank the various options based on a set of criteria, including:

- Capital cost,
- Operating cost,
- Reliability,
- Integration,
- Flexibility,
- Environmental,
- Social.

The initially ranked options were further evaluated based on a series of factors incorporating: the existing primary facilities, need to expand to the ultimate build-out scenario, the extent of the interim treatment requirements and the limited need for nutrient removal. The final ranking and options to be analyzed in detail were:

1A Primary treatment followed by 50% ADWF CAS
1B. Primary treatment followed by 100% ADWF CAS
2. Primary treatment followed by 50% ADWF RTF
3. 50% ADWF HRAS in parallel with primary treatment
4. CEP followed by 50% RTF
5. CEP applied to all flows.

5.3 ANALYSIS OF PROCESS OPTIONS THAT PASSED FIRST LEVEL OF SCREENING

Typical design values were used to develop a comparison between the six options which passed the first level of screening. This approach provided a consistent comparison level for each of the 6 options. Schematic process flow diagrams, conceptual plant layouts, footprint requirements, sludge production, effluent quality projections, capital and O&M cost estimates, process flexibility and other factors were developed and evaluated. The following is a brief summary of that assessment.

Option 1A: Primary treatment followed by 50% ADWF CAS

The primary treatment process will continue to treat 100% of the PWWF. The CAS process will be designed to provide treatment for 50% of ADWF (228 MLD) and 50% of MM BOD_5 loading (40 t/d) following primary treatment. The CAS treated flow and the untreated primary effluent would be mixed prior to the effluent pump station. Four aeration tanks and four clarifiers would be required to treated 50% of the ADWF. Additional solids handling units would be required for the additional biological solids, including, two DAF thickeners and two additional digesters. The total interim footprint requirements are estimated as 30,600 m^2 and the estimated capital cost is $124,620,000 not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

Option 1B: Primary treatment followed by 100% ADWF CAS

This option is the same as option 1A however the CAS is designed to treat 100% of the ADWF (456 MLD) and a BOD_5 load of 81 t/d. This option was retained before the bench scale testing indicated that 50% ADWF biological treatment would provide a significant improvement in toxicity testing. Eight aeration tanks would be required along with eight clarifiers, four DAF thickeners and four additional digesters. The total footprint for this option has been estimated as 52,200 m^2. The estimated capital cost for option 1B is $228,030,000 not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

Option 2: Primary treatment followed by 50% ADWF RTF

This process has similar flows and loads as option 1A, however a RTF would be used to provide biological treatment. Two trickling filters would be necessary along with four final clarifiers. To accommodate the additional solids two DAF and two digesters are necessary. The estimated footprint is 20,000 m^2 and the estimated capital cost is $112,500,000 not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

Option 3: 50% ADWF HRAS in parallel with primary treatment

In this process HRAS units are used in parallel with the primary treatment plant. A flow of 50% of ADWF and a BOD_5 load of 62 t/d would be applied to the HRAS. Three additional aeration tanks and four clarifiers would be required. Similar to the previous options four DAF thickeners and three additional digesters are required. A footprint of 22,500 m^2 and capital cost of $152,220,000 is estimated for this option not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

<u>Option 4: CEP followed by 50% RTF</u>

This option involves using CEP and treating 50% ADWF in a RTF. The flow to the RTFs would be 250 MLD with a BOD_5 load of 27.9 t/d. Two RTF would be required along with three clarifiers. To accommodate the increased in chemical sludge two additional gravity thickeners would be necessary along with one additional DAF. Four new digesters are necessary to handle the increased solids production. The necessary footprint is 22,700 m^2 for this option with an estimate capital cost of $123,800,000, not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

<u>Option 5: CEP applied to all flows</u>

This option only uses CEP treatment. Two additional gravity thickeners and three additional digesters would be necessary. The total footprint is estimated as 9,600 m^2 with a capital cost of $57,560,000 not including upgrading of the existing plant, mechanical dewatering and expansion to the cogeneration plant.

5.4 RESULTS OF SECOND LEVEL OF SCREENING

A second level of screening was undertaken to assess the merits of each of the six short listed options. The second level of screening was carried out using a multi-criteria analysis. The procedure used for the multi-criteria analysis is described in Section 10 of Appendix 3. In order to carry out this analysis, the options identified in Section 5.3 above where first analyzed in more details. This included unit process sizing and configuration development for each option based on typical design values and preliminary cost estimates.

A points weighted evaluation matrix was used to compare process, taking into account various elements specific to the IIWWTP site. The results of multi-criteria analysis are summarized in Table 5.1. The three major categories used for the evaluation were, cost and technical, environmental and social. Following the evaluation option 2, the 50% RTF was ranked highest based on the evaluation matrix due in part to the lower capital and operating costs and higher environmental ranking. Option 1A, 50% CAS was the second and CEP was assessed as the third ranking option.

Practical considerations, which must be assessed, involve the practicality of the interim option to be expanded to the build-out scenario. The TF option also had the top score in the ranking for the build-out to secondary, making it a preferred option for the interim period. Several additional attractive features with the TF option compared to conventional activated sludge are the smaller footprint, lower capital cost and lower operating costs. The final attractive feature of the TF option is the experience GVRD currently has with this process from the installations at Lulu Island, Annacis Island and NW Langley wastewater treatment plants.

TABLE 5.1
IONA ISLAND INTERIM TREATMENT
SUMMARY OF SECOND LEVEL OF SCREENING

	Option 1A		Option 1B		Option 2		Option 3		Option 4		Option 5	
	50% CAS		100% CAS		50% RTF		50% HRAS		CEP + 50% RTF		CEP	
	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking
Cost + Tech Weighted at 50%	79.7	2	68.2	6	81.9	1	77.5	4	69.5	5	78.7	3
Environmental Weighted at 50%	78.9	3	65.6	6	84.1	1	76.6	4	72.1	5	82.0	2
Social Weighted at 50%	81.7	1	73.6	5	77.8	3	78.4	2	64.8	6	77.0	4
Overall Rank	2		5		1		4		6		3	

Following the second level of screening, the following two options were selected for interim upgrades:

- Primary treatment + TF/SC for 50% of ADWF
- CEP + TF/SC for 50% ADWF with no secondary clarifiers

However, prior to finalizing the short list of options for interim upgrades for the Iona Island plant, the following activities were carried out:

- Forecast of effluent quality for TSS and BOD for the period from 2004 to 2021 for the existing primary plant based on flow and load projections described in Section 2.4.

- Forecast of effluent quality based on various interim upgrade options. These forecasts of effluent quality were also based on the flow and load projection described in Section 2.4.

The selection of upgrades options for interim treatment became an iterative process and other upgrade options were added to the two options identified above. The purpose of the iterations was to (1) identify the minimum upgrade required to meet permit to 2021 and, (2) to ensure flexibility and ease of phasing.

The short list of options for interim upgrades is described in Section 5.5. The forecast of effluent quality for the existing primary plant and various interim upgrade options is summarized in Section 5.6. The estimated cost for these options is described in Section 5.7 and proposed approaches to interim treatment are included in Section 5.8.

5.5 DESCRIPTIONS OF OPTIONS FOR INTERIM UPGRADES

The selection of unit operations is restricted to those, which are compatible with the Build-out to Secondary. The preferred process options for build-out to secondary are TF/SC and BAF. For the purpose of permit compliance to 2021, biological treatment was restricted to TF/SC since this process offers the greatest flexibility of phasing. In order to ensure compliance until 2021 either CEP treatment or partial biological treatment is assumed to be required. The use of combined CEP and biological treatment offers further improvement in effluent quality over partial biological treatment. The following interim upgrade options were assessed:

Option 1 - RTF for 25% of ADWF
Option 2 - RTF for 50% of ADWF
Option 3 - CEP
Option 4 - CEP + 50% ADWF RTF (no secondary clarifiers)

<u>Options 1 and 2 – Roughing Trickling Filter for a Portion of ADWF (25% and 50%)</u>

The primary treatment plant will be upgraded to include fine screens and will continue to treat the entire peak wet weather flow. The secondary RTF will treat a portion of the average dry weather flow (25% or 50%) and a portion of the maximum month load. Final clarifiers will be used to remove TSS and biological solids produced in the RTF. Flow greater than the capacity of the biological treatment plant will by-pass the secondary treatment and be recombined with secondary effluent prior to the ocean outfall. Biological and primary sludge will be thickened in DAF units and gravity thickeners, respectively. The thickened sludge will be mixed and stabilized in mesophilic anaerobic digesters.

<u>Option 3 – CEP</u>

The entire flow will receive preliminary and CEP treatment. However, CEP could be turned off when the effluent is highly diluted during periods of wet weather flow. In order to handle the additional sludge produced by the CEP process, additional gravity thickeners and digesters are required.

<u>Option 4 – CEP with 50% RTF (no secondary clarifier)</u>

Chemically enhanced primary, with 50% ADWF treated in a rough trickling filter is an alternate interim option. The primary plant will be upgraded to include fine screens and will continue to treat the entire peak flow. Following CEP treatment, 50% of the ADWF will be diverted to the RTF, which will be designed to treat 50% of the maximum month load (28 t/d BOD and 12 t/d TSS). No secondary clarifiers will be provided. The primary and secondary effluent will be recombined prior to ocean outfall. Primary sludge will be gravity thickened and stabilized using mesophilic anaerobic digestion. To address the issue of potential high suspended solids in the effluent resulting from sloughing from the trickling filter, the effluent from the RTF could be re-directed to the primary clarifiers.

The key differences between the various options in terms of number of components for the unit processes is summarized in Table 5.2. Option 1 does not provide process equipment redundancy because only one unit is required for several unit processes.

TABLE 5.2
UNIT PROCESS DESCRIPTION – INTERIM UPGRADES FOR IIWWTP

Unit Process	Interim 2021			
	Option 1 25% RTF	Option 2 50% RTF	Option 3 CEP only	Option 4 CEP + 50% RTF no SCL
Design Flow for Biological Treatment	114 ML/d	228 ML/d	0	228 ML/d
Additional Primary Sedimentation Tanks (PST)	0	0	0	0
Trickling Filter (TF) 44m dia. × 6 m high	1	2	0	2
Solids Contact (SC) 78 m × 18 m × 5 m	0	0	0	0
Biological Aerated Filters (BAF) 14 m ×10 m × 5 m	0	0	0	0
Secondary Clarifiers (SCL) 41 m dia. × 5 m	2	4	0	0
Gravity Thickeners (GT) 20 m dia. × 4 m	0	0	2	2
Dissolved Air Flotation (DAF) 21.5 m dia. × 3.5 m	1	2	0	0
Digesters 32m dia. × 10.6 m	1	2	3	3
Centrifuge 145 m^3/hr	2	3	3	3

5.6 FORECAST OF EFFLUENT QUALITY

5.6.1 Methodology

The future performance of the primary plant was estimated in order to determine when the effluent BOD and TSS levels of 130 and 100 mg/L respectively would exceed the Operational Certificate concentrations. Compliance was assumed to be 99% percentile for both BOD and TSS.

The estimations were based on historic removal efficiencies, SOR, HRT and loading rates. Two separate evaluations were used to assess the future performance,

1. Average annual effluent based on annual average flow and loading conditions,
2. Frequency of effluent concentrations exceeding the permit levels.

Existing removal efficiency as a function of various sedimentation tank properties, such as SOR, HRT, weir loading, VSS:COD, BOD:COD and VSS:TSS ratios, were compared to isolate the most sensitive factors influencing removal efficiencies. The SOR was determined to have the greatest influence on removal efficiency.

The forecast of effluent quality was carried out using the second method above. The assessment was based on an evaluation of the frequency of events in which the permit concentration was exceeded. This evaluation was based on the following methodology,

- The average effluent BOD and TSS concentration frequency for the years 2000 to 2002 was used as a baseline,
- Influent BOD_5 and TSS levels were estimated based on the average flows and maximum month loads,
- SOR were calculated based on the historic performance and projected flow rates,
- The frequency of effluent concentrations for the years 2004, 2011 and 2021 was calculated.

Based on the TSS concentration frequency analysis the current compliance level of 99.5% will be reduced to 98% by the year 2021. The BOD_5 concentration compliance was calculated to be reduced from 97% to 80% for the years 2004 and 2021, respectively.

It should be noted that the percentile of occurrence analysis is one of the results-based approaches, which includes all probable factors in the collection and treatment system. The percentile occurrence distribution can be impacted by a variety of factors, including a future hydraulic upgrades (i.e. flow split improvements), water conservation, waste source control and possibly storm events which may increase or decrease the frequency of effluent concentrations exceeding the permit levels.

5.6.2 Compliance Level Projections

Compliance projections for the existing plants and for a number of interim upgrade options are shown on Figure 5.1 for TSS and Figure 5.2 for BOD. Compliance has been defined as 99% or better reliability to achieve the required effluent concentration.

As shown on Figures 5.1 and 5.2, the existing primary plant will not meet compliance level for BOD in 2004 and will not meet compliance for TSS in 2009. It should be noted that this analysis is based on the following assumptions:

- Loading (TSS and BOD) projections for the maximum month (see figures 2.4 and 2.5)
- Flow projections based on average annual flow projections.
- Removal rate based on existing plant performance.

All four interim upgrade options will meet TSS compliance level until 2021 when the plant is upgraded to build-out to secondary. With interim upgrade Option1, the plant would meet compliance for BOD until 2015. With interim upgrade Options 2, 3 and 4, the plant would meet compliance until 2021.

FIGURE 5.1
TSS RELIABILITY LEVEL PROJECTIONS FOR IIWWTP

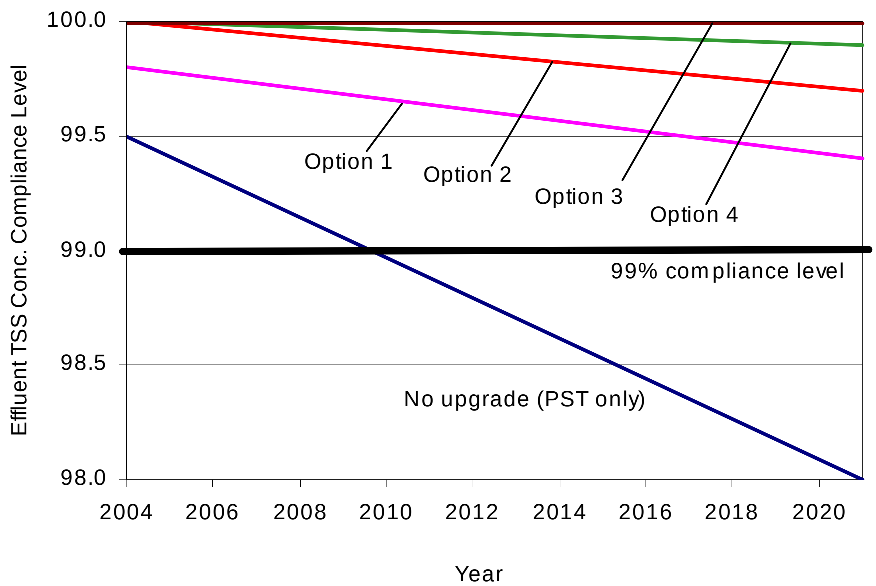
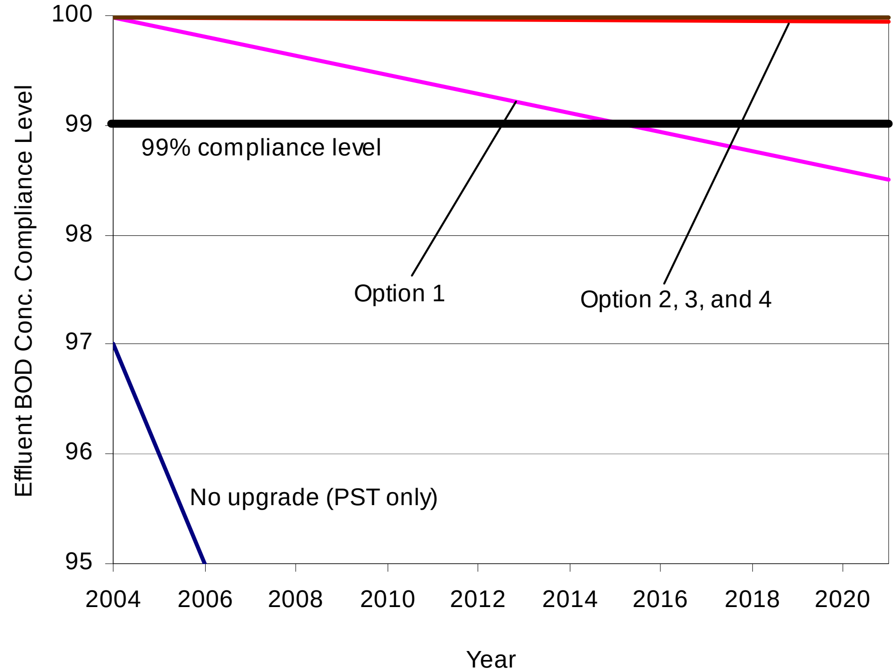


FIGURE 5.2
BOD RELIABILITY LEVEL PROJECTIONS FOR IIWWTP



5.7 SUMMARY OF ESTIMATED COST FOR PREFERRED OPTIONS

Following additional modeling to refine the sizing of the various unit processes, revised cost estimates were developed for the two preferred options. The following items were also added: mechanical sludge dewatering, expansion of cogeneration plant to provide capacity for additional gas generation, upgrading of existing plant by adding fine screens. Based on concept drawings, the cost of site improvements was also revised.

The assumptions and parameters used for estimating the costs are discussed in Section 4.7, with the following parameters applicable to interim:

- Evaluation period for interim: 2004 to 2020
- Construction period for interim: 2008 -2009 (1/2 of capital each year)
- Commissioning date for interim: 2010

TABLE 5.3
CAPITAL COST ESTIMATES – INTERIM UPGRADES

YEAR	Interim 2021			
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL
CAPITAL COSTS				
Site Improvements	$8,865,000	$21,775,000	$3,030,000	$13,046,000
Chemical Feed	$0	$0	$1,500,000	$1,500,000
Biological Aerated Filter	$0	$0	$0	$0
RTF/TF/SC	$8,395,000	$16,790,000	$0	$16,790,000
Solids Contact Tank	$0	$0	$0	$0
Secondary Clarifiers (SCL)	$5,649,600	$11,299,200	$0	$0
Gravity Thickeners	$0	$0	$2,772,000	$1,935,000
Sludge Blending Tank	$250,000	$500,000	$0	$0
DAF Thickeners	$7,695,600	$15,391,200	$0	$0
Digesters	$8,013,500	$16,027,000	$24,026,400	$24,026,400
Mechanical Dewatering	$7,000,000	$10,000,000	$10,000,000	$10,000,000
Site Works	$2,847,500	$4,002,500	$150,000	$3,135,000
Admin/Maint. Building	$0	$0	$0	$0
Control System	$1,855,000	$3,711,000	$1,593,000	$2,150,000
Expansion of Cogeneration	$1,500,000	$7,000,000	$9,900,000	$9,900,000
Odour Control	$500,000	$1,000,000	$0	$1,000,000
Existing Facility Upgrades	$11,000,000	$11,000,000	$11,000,000	$11,000,000
Sub-Total	$63,571,000	$118,496,000	$63,971,000	$94,482,000
Division 1 Cost	$1,368,000	$2,418,000	$1,524,000	$2,036,000
Engineering	$10,171,000	$18,959,000	$10,235,000	$15,117,000
Project Management/QA/QC	$2,543,000	$4,740,000	$2,559,000	$3,779,000
Contingency	$19,071,000	$35,549,000	$19,191,000	$28,345,000
Total Capital Costs	$96,724,000	$180,162,000	$97,480,000	$143,759,000

TABLE 5.4
OPERATING AND MAINTENANCE COST ESTIMATES FOR INTERIM UPGRADES FOR IIWWTP

YEAR	Interim 2021			
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL
O&M COSTS				
Labour	$4,359,000	$4,695,000	$4,359,000	$4,695,000
Chemical Costs*	$0	$0	$7,847,000	$7,847,000
Residuals Management	$4,091,000	$4,888,000	$5,329,000	$5,329,000
Energy/Power	$1,136,000	$1,207,000	$1,188,000	$1,213,000
Repair/Maintenance	$3,550,000	$4,218,000	$2,930,000	$2,776,000
Administration and others	$1,671,000	$1,720,000	$1,671,000	$1,699,000
Total (O&M Costs)**	$14,805,000	$16,727,000	$23,323,000	$23,557,000
Total (O&M Costs)***	$5,714,000	$7,635,000	$14,231,000	$14,465,000

Notes
*: Polymer for dewatering is not included in Interim Option 1 and Option 2
**: Entire plant O/M costs including existing primary plant and upgrade
***: Upgrade O/M costs only (existing primary plant excluded)

TABLE 5.5
LIFECYCLE COST FOR INTERIM UPGRADES FOR IIWWTP

YEAR	Interim 2021			
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL
Discounted Total O&M Cost	$31,770,000	$42,451,000	$79,124,000	$80,425,000
Discounted Capital Costs	$70,233,000	$130,818,000	$70,782,000	$104,385,000
Total Capital and O & M Costs at Present Value	$102,002,000	$173,268,000	$149,905,000	$184,810,000

5.8 APPROACHES TO IMPLEMENTATION

The selection of the preferred options for the interim upgrade has to consider the following factors:

- Final process option selected for the build-out to secondary
- Construction date for build-out to secondary
- Permit reliability for BOD and TSS
- Improvement in LC_{50} bioassay test results

TABLE 5.6
SUMMARY OF ANALYSIS FOR INTERIM UPGRADES

Interim Option	Effluent BOD & TSS Reliability above 99%	Improvement in LC_{50} Test	Remarks
TF/SC for 25% of ADWF	To 2021 for TSS To 2015 for BOD	Minimum improvements (based on small scale testing)	• 1/8 of build-out to secondary plant capacity. • Similar capital cost to option 3 ($96.8M vs. $97.5M). • Lowest LCC.
TF/SC for 50% of ADWF	To 2021 for TSS and BOD	60% (based on small scale testing)	• 1/4 of build-out to secondary plant capacity. • Highest capital cost ($180.2M).
CEP only	To 2021 for TSS and BOD	60% (based on small scale testing)	• Can be operated intermittently • Potentially generates largest quantity of sludge. • Similar capital cost to Option 1 ($97.5M vs. $96.8M), however higher LCC than Option 1. • Allows postponement for the selection of biological process. • Very high cost of chemical.
CEP + TF/SC for 50% of ADWF and no secondary clarifier	To 2021 for TSS and BOD	60% (based on small scale testing)	• Provides flexibility. • Potentially generates largest quantities of sludge. • Less expensive than Option 2 ($143.8M vs. $180.2M), however, highest LCC.

The summary of the analysis for the various upgrades options provides an indication of three possible paths for interim upgrades to 2021:

- Partial biological treatment using TF/SC to treat 25% or 50% of ADWF
- Combination of CEP and partial biological treatment
- CEP

6.0 SOLIDS HANDLING

This chapter summarizes the average annual sludge quantities estimated for interim options for IIWWTP as discussed in Appendix 3 and Appendix 7 (Sections 6.1). These summaries are useful for process comparison. Updated sludge volume estimation is also included in Section 6.6, which is the outcome of preferred options as discussed in Appendix 10. This is developed to assist the GVRD in evaluating options for sludge disposal and reuse. Options for sludge pre-treatment are presented in Section 6.2 while options for sludge stabilization to produce a Class A product are presented in Section 6.4.

6.1 ESTIMATED SLUDGE QUANTITIES AND QUALITY

The sludge quantity and characteristics are critical parameters in the decision to select the secondary treatment upgrade processes. Different treatment processes will produce different types of sludge. The type of sludge produced will impact the required level of treatment, capital cost investment, O/M costs and reuse options. The following factors are the main considerations in the assessment of the treatment processes:

- Sludge quantity
- Ease of sludge stabilization (i.e. digestion)
- Ease off handling (e.g. dewaterability)
- Nutrient values for land application or other recycling options
- Metals content

The quantity of sludge produced at the IIWWTP for each of the five interim process options was compared by developing sludge estimates based on the average annual BOD and TSS loading in order to compare the annual sludge production. The sludge quantity, express as undigested dry solids mass and a wet volume after dewatering, are presented graphically in Figures 6.1 and 6.2. Process option 4, CEP + 50% RTF produces the highest sludge mass, while process option 1B, Primary + 100% CAS produces the highest wet volume.

An assessment of metal content was conducted on the primary sludge and estimated. Metal levels in the CEP sludge and biological sludge is expected to be higher than in the primary due to the presence of potential metal coagulants (from CEP treatment) and biological absorption. A summary of estimated metal quality in the sludge is presented in Table 6.1.

By 2021, the screening and grit are estimated to be about 320 Tonnes/year and 1,700 Tonnes/year, respectively, assuming no screening and degritting upgrade during the interim stage. In 2003, total screening and grit wet weight are about 226 and 1,580 tonnes/year, respectively. Grit and screening production at Iona Island has been steadily decreasing over the last four years. A substantial increase in screenings and grit are expected in the process upgrades (e.g. fine screens and grit removal) are implemented.

FIGURE 6.1
PROJECTED SLUDGE QUANTITY (UNDIGESTED DRY SOLIDS MASS) OF IIWWTP

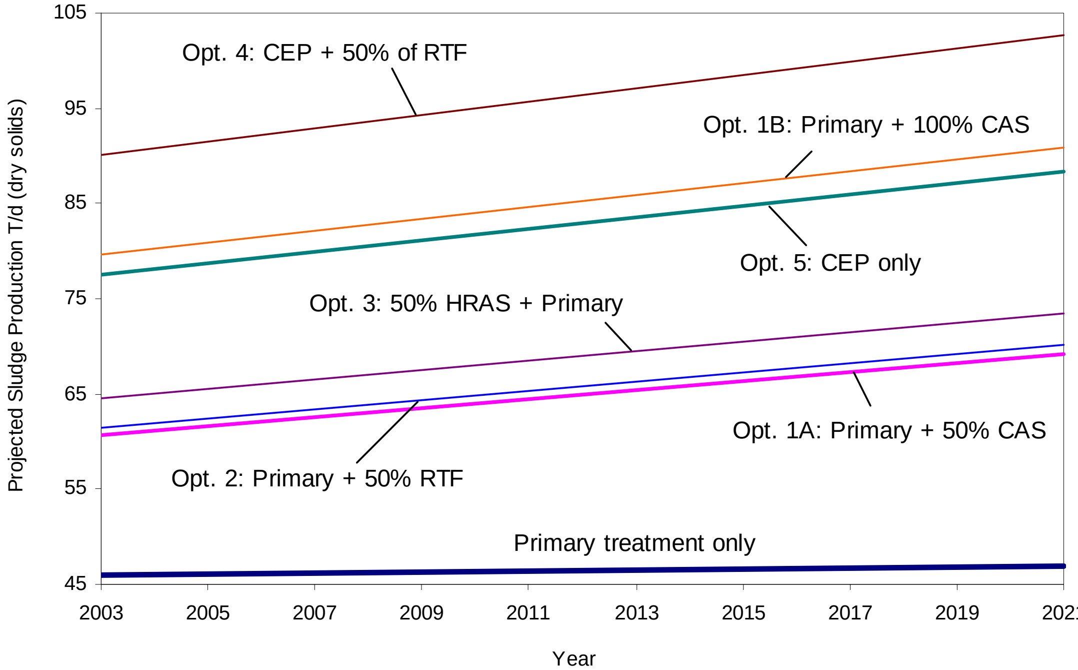


FIGURE 6.2
PROJECTED SLUDGE QUANTITY (WET VOLUME AFTER DEWATERING) OF IIWWTP

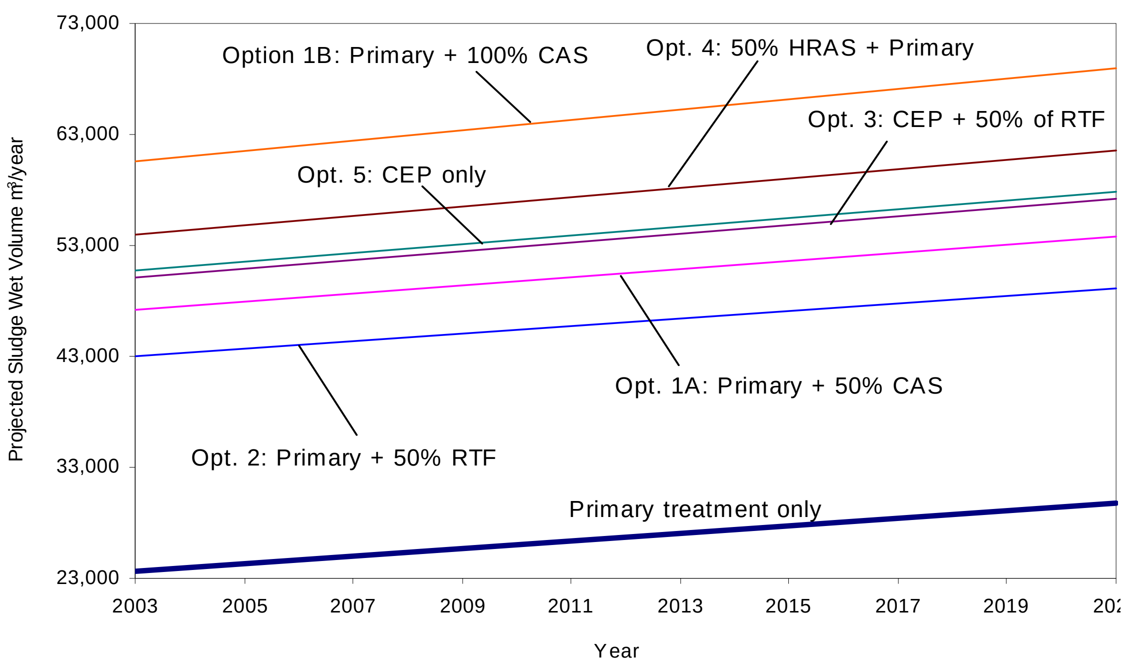


TABLE 6.1
ESTIMATED SLUDGE / BIOSOLIDS QUALITY

Chemicals/Nutrients (mg/kg dry kg)	Primary Sludge	CEP Sludge*	Secondary Sludge*	Digested Sludge**
Arsenic Total	1~3	N/A	5~10	8
Cadmium Total	1~3	10~20	5~10	3.2
Chromium Total	30~70	300~400	100~150	71
Cobalt Total	2~5	N/A	5~10	4.7
Copper Total	1,000~1,800	3,000~4,000	2,000~3,000	1,360
Lead Total	60~90	400~600	150~200	91
Mercury Total	5~8	N/A	5~8	3
Nickel Total	30~50	100~200	50~100	19
Zinc Total	400~700	1,000~2,000	700~1,500	908
Total Nitrogen	25,000~40,000	28,000~45,000	30,000~50,000	~55,000
Total Phosphorus	10,000~20,000	15,000~25,000	20,000~30,000	N/A

*: in part based on Bonnybrook WWTP, Calgary, 1998
**: Annacis Island WWTP digested combined sludge (2002)

6.2 OPTIONS FOR SLUDGE PRE-TREATMENT

Sludge pre-treatment is an option that can improve the sludge digestion rate and extend the capacity of new and existing digesters. Appendix7 discussed nine options for pre-treatment. Results of the initial assessment short- listed three viable technologies. These three sludge pre-treatment options are briefly discussed below. Further engineering and economic evaluations are required prior to selecting any of these options. However, the cost of Ultrasonic sludge pre-treatment has been included in the capital cost estimates for build-out to secondary.

- Ultrasonic treatment utilizes an ultrasound frequency of 20 kHz to rupture the biomass cellular wall. This results in a more soluble substrate, an increase in the digestion rate, biogas production, dewaterability and filamentous control. Sonix™ is marketed in North America by Sonico. The Avonmouth, UK, wastewater plant has operated with Sonix™ for the past three years. Full-scale testing at the Orange County WWTP, California, resulted in a 30% increase in VS reduction. Additional full-scale testing is currently underway at Edmonton, AB and Mangere, New Zealand.

- Alkaline treatment, or MicroSludge™, is marketed by Paradigm Environmental Technology and is a thickened waste activated sludge pre-treatment. Sodium/potassium hydroxide is used to elevate the sludge to a pH greater than 10 for a period of 60 minutes. Mechanical shear force and sudden pressure relief weaken cellular walls and ultimately result in rupture. The sludge pH must be neutralized prior to digestion; otherwise ammonia toxicity will impact the anaerobic microorganisms. The MicroSludge™ process has been pilot tested at the Lulu Island WWTP and resulted in a reduction in the digester HRT from 15 to 5 days. A full-scale application is currently being tested at the Chilliwack WWTP.

- Thermal pre-pasteurization is a process in which the sludge is treated for a minimum of 30 minutes at a temperature of 70°C prior to mesophilic digestion. Typically the mesophilic digestion stage is 15 days. The primary advantage of pre-pasteurization is to achieve high rate of pathogen kills but it will not reduce sludge volumes. Several full-scale applications exist in North America, including: JAMES plant Abbotsford, Perris, California and Franklin, Pennsylvania. No additional digester capacity is gained from the pre-pasteurization process, and as such digester capacity increase is inevitable.

6.3 SLUDGE STORAGE LAGOON

The existing lagoons are utilized for dewatering of the digested sludge. Following the expansion to the interim option it is estimated that lagoons will exceed the typical loading of 0.25 kg VSS/m^2/d within 4 to 8 years. Additionally, lagoon #1 will be used for the solids treatment plant expansion. Overloading the remaining lagoons will result in a reduction in the solids settling efficiency, reduced supernatant quality (both TSS and BOD), large stockpiling volume, more frequent dredging, and reduced pathogen reduction. Dewatering of the biological sludge following digestion will be lower than that of the current primary sludge, further reducing the lagoon performance. It is recommended that mechanical dewatering be phased in to complement the lagoons as the sludge volume increases.

6.4 SLUDGE STABILIZATION

Several options have been reviewed which could be implemented at the IIWWTP to achieve a Class A biosolids, including: temperature phased anaerobic digestion, acid-gas anaerobic digestion, and extended anaerobic digestion. These options are briefly discussed below; details are presented in the Appendix 7.

- Temperature phased anaerobic digestion (TPAD) utilizes both mesophilic (~35°C) and thermophilic (~55°C) digestion in series. The thermophilic process, typically 5 days SRT, alleviates the foaming and odour problems typically associated with single stage operation. The subsequent mesophilic process, typically 10 days SRT, is shorter than the normal 15 to 20 SRT for a single stage process. The overall SRT is reduced and increase gas production is viable.

- Acid-gas anaerobic digestion (AGAD) is a two-phase process which utilizes a low pH (<6) short SRT (1 to 3 days) in the first stage to achieve a solubilization of the particulate material into volatile acids. The second stage is operated at a neutral pH and a longer SRT (10 to 15 days) for optimal gas production. Various combinations of AGAD and mesophilic or thermophilic operation exist. Wood-Green Valley WWTP, Illinois currently operates an AGAD system with thermophilic-mesophilic digestion.

- Extended thermophilic anaerobic digestion (ETAD) is a two or three stage process where multiple tanks are operated in series. VS reduction is best achieved in a completely mixed tank, while pathogen reduction is best achieved under plug flow conditions. The ETAD process results in higher reaction rates, smaller digesters, high VS destruction, higher gas production, higher pathogen kills, and reduced foaming. Substantial process upgrades are required to operate the digesters at IIWWTP ETAD mode.

6.5 SUMMARY OF RECOMMENDATION FOR INTERIM SLUDGE MANAGEMENT

The current process of utilizing the lagoons for storage/settling and on-site stockpiling are considered the most economical options. However, a portion of the lagoon space must be utilized for plant expansion and the increase in sludge volume will result in eventual over loading of the lagoons (4-8 years). Alternative dewatering options were examined for the interim stage, these include:

1) Operate lagoons and haul the sludge off-site on a yearly basis using dredging and mobile centrifuges. This option will result in almost double the sludge volume compared to the current process due to the lower dewatering efficiency of the centrifuges.

2) Continued operation of the lagoons and stockpiling of sludge on site.. This option will result in an increase of the loading rate as sludge volume increases and lagoon area decreases. Recycled supernatant quality will be reduced and increased land area for stockpiling will be required, impacting the existing wetlands.

3) Abandon the lagoons and install complete mechanic dewatering. Limited on-site storage will be required and a higher nutrient value is achieved compared to the lagoon process. Higher capital and O/M costs are associated with this option.

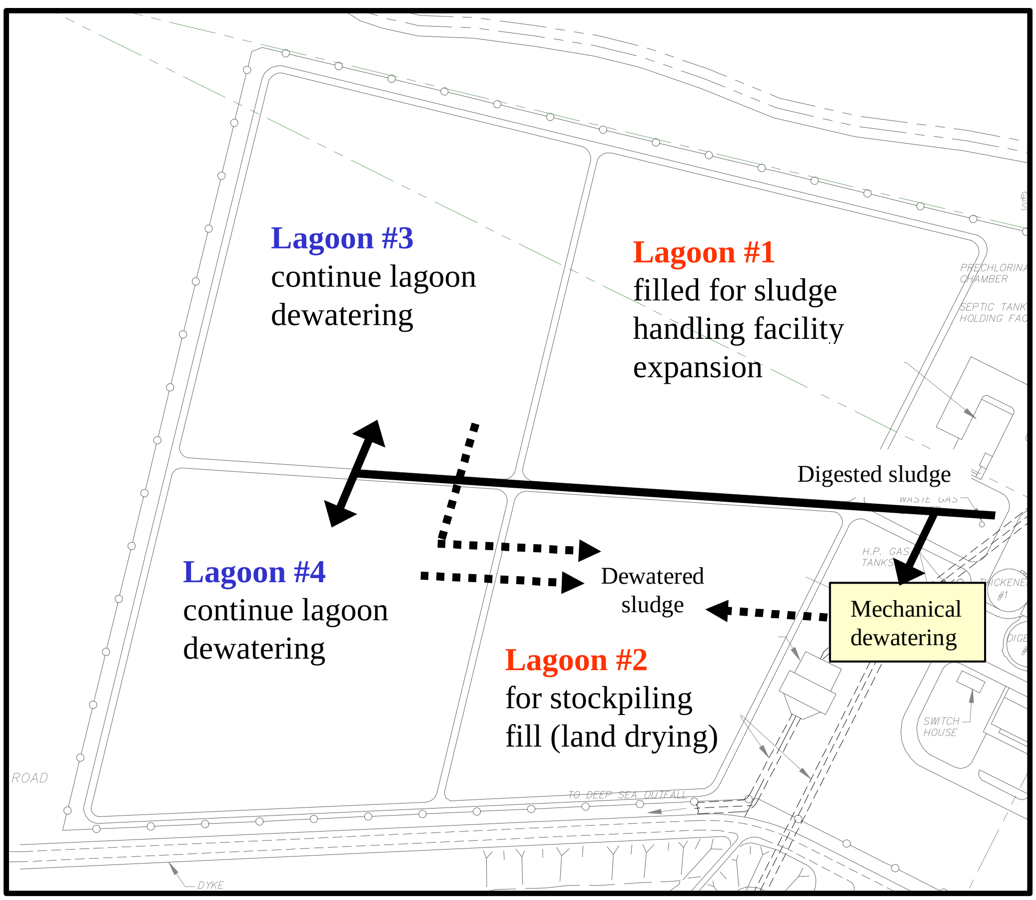
4) Phase out the lagoons by implementing mechanical to dewater additional sludge produced by the interim upgrade. This approach will defer the full capital costs associated with the installation of mechanical dewatering equipment, while maximizing the use of the existing facilities.

Option 4 provides is the most flexible option to achieve different qualities of sludge and potential sludge end products. It should be noted that one of the lagoons will be needed to provide space for the interim and future plant expansion and a second will be used to stockpile the dewatered sludge. The remaining lagoons would be operated at an optimal VS loading rate, the remaining sludge would be mechanically dewatered using centrifuges. A schematic arrangement of the interim lagoon operation is presented in Figure 6.3.

In summary, for interim sludge treatment/management at IIWWTP, the following planning approaches are recommended:

- Improve and optimize the gravity thickener and digester performance.
- Continue to operate the digester under mesophilic anaerobic conditions, lagoon land drying, and stockpiling, until the interim upgrade is implemented.
- In order to provide space for interim upgrade, one lagoon cell to be emptied in 2006 in order to proceed with preloading in 2007.
- Remove part of the stockpiles of sludge east of the existing plant in 2006 in order to proceed with preloading in 2007.
- Add additional digesters with the capability to be operated at both mesophilic and thermophilic modes.
- Add mechanical dewatering facility to compensate for the deficiency of lagoon capacity, and phase in centrifuges as part of the interim upgrade.
- Consider sludge pre-treatment options for waste activated to defer digester expansion. Sludge pre-treatment applies only to waste activated sludge produced by secondary treatment.

FIGURE 6.3
RECOMMENDED INTERIM SLUDGE HANDLING AT IIWWTP



6.6 UPDATED SLUDGE QUANTITIES FOR PREFERRED TREATMENT OPTION

Sludge volume and mass estimates vary depending on the selected treatment option and whether or not CEP treatment is used. Two options were selected for the plant expansion, 50% RTF with or without CEP for the interim period and TF/SC for the build-out to secondary process. Table 6.2 summarizes the projected sludge volumes and masses for these options.

The sludge volume assessments presented in Appendix 4 were based on maximum month loads for design purposes (unit process sizing). The sludge projections presented in Table 6.2 and in Appendix 10 are based on annual average loads.

TABLE 6.2
ESTIMATED SLUDGE PRODUCTIONS FOR PREFERRED OPTIONS

YEAR		Interim 2021				Build-out to Secondary 2036	
Option	Unit	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Raw Sludge/Biosolids							
Primary Sludge	t/d	48	48	-	-	50	50
CEP Sludge	t/d	0	0	91	91	0	0
Secondary Biosolids	t/d	12	23	0	0	42	56
Total Raw Sludge	t/d	60	72	91	91	92	105
Thickened Sludge							
Gravity Thickener	m^3/d	967	967	1,777	1,777	992	992
DAF Supernatant	m^3/d	334	669	0	0	1,202	1,591
Total Thickened Sludge	m^3/d	1,301	1,635	1,777	1,777	2,193	2,583
Digested Sludge	m^3/d	1,301	1,635	1,777	1,777	2,193	2,583
Dewatered Sludge	m^3/d	112	134	146	146	190	218

PART 2 – UPGRADING OF LIONS GATE WWTP

7.0 ANALYSIS AND EVALUATION CRITERIA

7.1 EFFLUENT CRITERIA

7.1.1 Liquid Waste Management Plan Requirements

The Liquid Waste Management Plan (LWMP) has indicated that the base level of treatment for the Lions Gate treatment plant should meet the following maximum daily concentration levels (Operational Certificate ME-00030):

- BOD_5 130 mg/L
- TSS 130 mg/L

The above concentrations are based on flow proportional 24-hr composite samples. The LWMP further indicates that the Lions Gate treatment plant will provide secondary treatment for flows up to two times dry weather flow by the year 2031. One of the commitments of the LWMP is to upgrade the plant by adding facilities for chemical addition if necessary in order to meet the above effluent concentrations. Another commitment of the LWMP is to upgrade the level of treatment if enhanced primary treatment is not adequate to address environmental concerns and to maintain effluent concentrations and loadings.

7.1.2 Municipal Sewage Regulation

The LWMP does not contain a definition of secondary treatment nor does it include effluent criteria for the Lions Gate plant. The ***BC Municipal Sewage Regulation*** includes the following definition of secondary treatment and the effluent criteria:

Secondary treatment – any form of treatment, excluding dilution, that consistently produces an effluent quality with a BOD_5 not exceeding 45 mg/L and TSS not exceeding 45 mg/L for flows up to 2.0 x ADWF.

7.2 FLOWS AND LOAD SCENARIOS

7.2.1 Methodology and Definitions

The same methodology for loads and flow projection was followed for Lions Gate and for Iona Island. Detailed flow and load projections were carried out in order to generate a lower and upper envelope as well as a design case. Separate flow and load projections were prepared for the various contributors, namely:

(1) residential,
(2) commercial and institutional,
(3) industrial, and
(4) groundwater infiltration.

For each contributor, lower and upper growth rates were established and the impacts of various source control measures were estimated. Lower and upper envelopes for flows and loads at the plant were prepared by adding the lower and upper envelopes for the above four components. A similar procedure was used for deriving the design case.

The definition of key terms are described in Section 2.1.1

7.2.2 Summary of Historic Data

The Regional Utility Planning Division of the GVRD has established a per capita ADWF of 518 L/c/d for LGWWTP based on historic average unit flows from 1991 to 1999. These are presented in Table 7.1. Table 7.2 presents the key characteristics of the wastewater received by the Lions Gate WWTP historically.

TABLE 7.1
LGWWTP FLOW DISTRIBUTION BASED ON ADWF (518 L/C/D)

Source	Distribution (2002)	Flow, l/c/d	Flow, Ml/d
Residential	52%	270	47
C&I	11%	55	9.7
Industry	7%	35	6.0
Groundwater Infiltration	30%	158	27
Total	100%	518	90

TABLE 7.2
LGWWTP HISTORIC WASTEWATER CHARACTERISTICS

All sources	BOD g/c/d	TSS g/c/d	Data Source
Average Annual (AA)	77	88	Based on average historical data from 1991 to 1999
Max. AA	86	104	BOD: Based on 1996 data TSS: Based on 1998 data
Min. AA	61	68	Based on 1991 data

7.2.3 Flow and Load Projections for Various Scenarios

The flow and load projections for various scenarios are summarized in Table 7.3. The assumptions made in deriving the lower and upper envelopes as well as the most design case are described later in Section 7.3 of this report.

TABLE 7.3
LGWWTP FLOW AND LOAD SCENARIOS

	Existing	**2031 – Design Year for Interim Upgrades**			**2046 – Design Year for Build-out to Secondary**		
		Lower Envelope	Upper Envelope	Design Case	Lower Envelope	Upper Envelope	Design Case
ADWF (Ml/d)	91	90	116	104	91	131	111
PWWF (Ml/d) See note	307	297	378	337	297	420	356
Max Month BOD (t/d)	18	21	26	25	23	30	28
Max Month TSS (t/d)	22	25	31	28	27	36	32

Note: The peak wet weather flow into the plant is based on the capacity of the existing influent pump station

The following comments are provided based on Table 7.3:

- Increase in ADWF is minimal from Year 2031 to 2046.
- For BOD loading, there is marginal difference between the upper envelope and the design case for the years 2031 and 2046.
- The design case for ADWF takes into account the impact of existing water conservation measures. The upper case does not take these into account.

7.3 ANALYTICAL ASSUMPTIONS

Lower and Upper Envelopes

A summary of the flow and load parameters on which the lower and upper projection envelopes are based is as follows:

(1) Population projections are estimated in accordance with the population upper and lower ranges developed by the GVRD's Regional Development Division for the NSSA as indicated in Table 7.4.

(2) The upper envelope for flow assumes no new source control measures and increase in infiltration by 5%. The lower envelope for flow assumes the adoption of the "enhanced water conservation" initiatives and a 10% decrease in infiltration as a result of the sewer repairs.

(3) The lower and upper growth rates for commercial and institutional (C&I) flows and loads are based on the same growth rates as population.

(4) The upper and lower projection of residential and C&I load are based on food waste discharge. Upper envelope of residential load assumes 80% of new households would be equipped with food grinders, while lower envelope assumes food grinders in residential households are reduced from one third of all households to 10% of all households in the design year.

(5) The lower envelope for industrial flows and loads is based on the same growth rate as the lower range for population projection. The upper envelope assumes existing businesses to grow at 50% of population growth and new businesses to grow at the upper growth rate of population.

Design Case

Flow and load parameters developed in this study for the design case projection are based on the following assumptions:

(1) Population projection is estimated at lower envelope values plus 80% of the difference between the upper and lower ranges as indicated in Table 7.4.

(2) Flow projection for groundwater infiltration is estimated at lower envelope values plus 80% of the difference between the upper and lower envelopes.

(3) Increases in commercial and institutional flows and loads are based on the same growth rates as population as in (1).

(4) Residential and C&I loads assume the same growth rate as the upper envelope.

(5) Per capita residential flow is 243 L/cap/day based on existing water conservation measures.

(6) Industrial flows and load projections are the same as upper envelope.

Scenario Uncertainties

The population scenarios are indicated in Table 7.4. Upper and lower envelope populations are estimated by the Regional Development Division in the GVRD. Changes within the population scenario will significantly impact the flow to the treatment plant.

TABLE 7.4
LGWWTP POPULATION SCENARIOS

Year	Lower Envelope	Upper Envelope	Design Case
2001 – Existing	173,750		
2021	200,000	220,000	215,000
2031	215,000	244,000	237,000
2046	241,000	285,000	275,000
2051	250,000	300,000	289,000

In conjunction with the variability in population growth, the other significant factor in estimating flows is the impact of water conservation measures. The existing water conservation program is assumed for the most probable upper case scenario, while the lower envelope assumes "enhanced" water conservation initiatives. Table 7.5 illustrates the per capita flow with these assumptions.

TABLE 7.5
LGWWTP RESIDENTIAL AND COMMERCIAL
PER CAPITA FLOWS SCENARIOS (L/cap/day)

Year	Lower Envelope	Upper Envelope	Design Case
2001 – Existing All sources	• Residential (Res.): 270 • Commercial (Com.): 55		
2021	• Res.: 232 • Com: 51	• Res.: 270 • Com: 55	• Res.: 243 • Com: 55
2036	• Res.: 232 • Com: 51	• Res.: 270 • Com: 55	• Res.: 243 • Com: 55

Variability in the load projection is mainly based on the implementation of the control of food waste discharges for residential and C&I sectors. Reduction in food waste discharge due to reduction of garburators in households will account for 5% and 8% decreases in BOD and TSS per capita load in 2046 respectively.

The variability in load and flow projections is shown on Figures 7.1 to 7.3

FIGURE 7.1
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASE SCENARIOS FOR ADWF

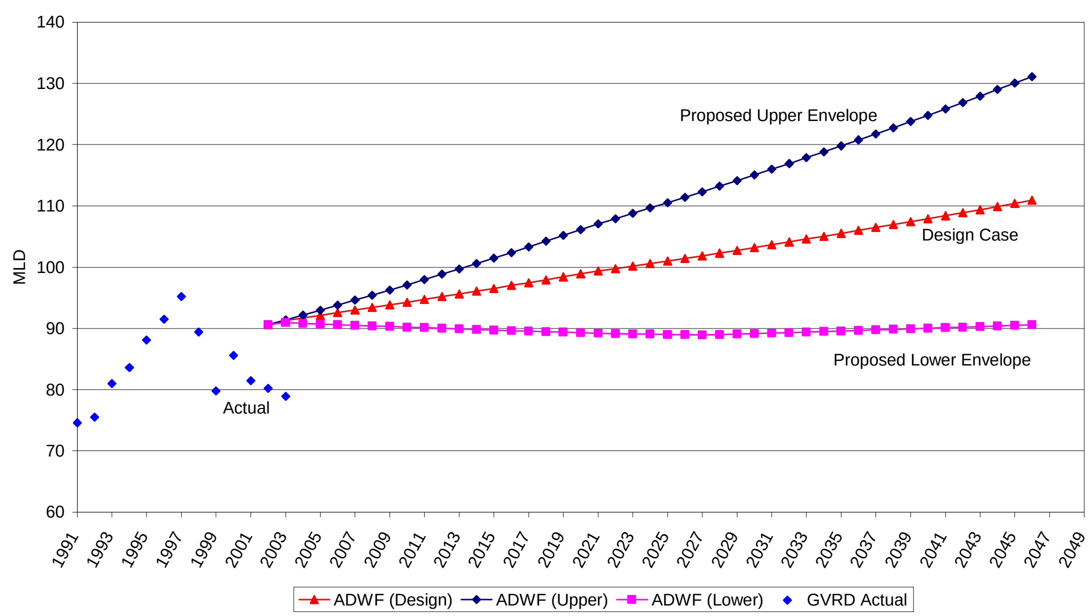


FIGURE 7.2
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASE FOR BOD (MAX. MONTH)

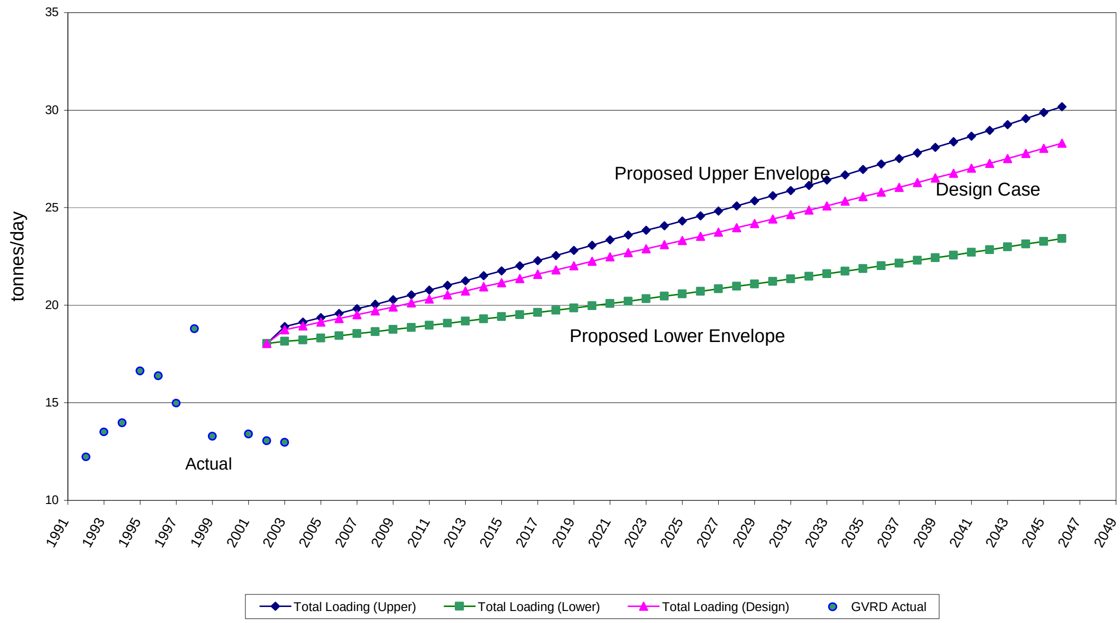
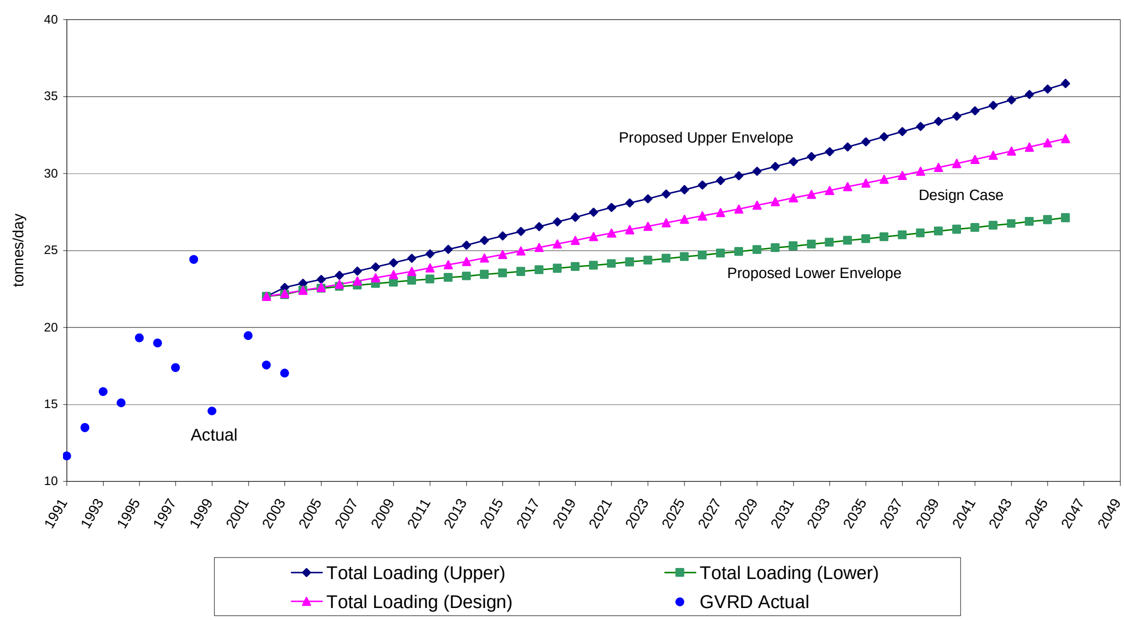


FIGURE 7.3
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASE FOR TSS (MAX. MONTH)



7.4 PROPOSED ANALYTICAL SCENARIOS AND CRITERIA

7.4.1 Analytical Scenarios

The design flows and loading values for the Lions Gate WWTP are indicated in Table 7.6. The upper envelope values are considered for Appendix 3. However, the design case values will be used for design of final options (Appendix 10).

TABLE 7.6
PROPOSED DESIGN LOADS AND FLOWS FOR LGWWTP

Year	Upper Envelope		Design Case	
	2031	2046	2031	2046
ADWF (Ml/d)	116	131	104	111
PWWF* (Ml/d)	378	420	337	356
BOD (t/d)	26	30	25	28
TSS (t/d)	31	36	28	32

*: The GVRD's commitment in the Liquid Waste Management Plant (LWMP) is to treat 2×ADWF. The intension is therefore to manage I&I and wastewater flows to limit the peak flow to approximately 2×ADWF. The valves shown in this table are therefore theoretical.

7.4.2 Proposed Design Criteria

The proposed design criteria for the Lions Gate wastewater treatment plant upgrade are indicated in Table 7.7

TABLE 7.7
PROPOSED DESIGN CRITERIA FOR LGWWTP

Parameter	Interim Upgrades	Secondary Treatment
Design year	**2031**	**2046**
Design Flow to Primary Treatment (Ml/d)	337*	356*
Design Flow to Secondary Treatment (Ml/d)	52**	222
Effluent Standard BOD (mg/L)	130	45 (20)***
Effluent Standard TSS (mg/L)	130	45 (20)***

Notes
*: The impact of future I&I reduction programs have not been considered in determining the peak wet weather flows. I&I reduction program should limit or eliminate the need to expand headwork capacity and limit the peak flows to the PSTs.
**: Treatment of 50% of ADWF assumed.
***: Design target (45 mg/L on maximum day and 20 mg/L for average annual)

7.5 PERMIT COMPLIANCE ISSUES TO 2031

7.5.1 General

Primary sedimentation tanks (PST) are operated to remove substantial portions of readily settleable solids and organic substrates associated with solids. An efficient PST system is capable of removing 50~70% of total suspended solids (TSS) and 25~45% of biochemical oxygen demand (BOD). However, the removal efficiency is subject to many factors, and many of these factors could be combined.

In addition, some specific operating conditions upstream of the PST will also affect the PST performance. At Iona Island these specific factors include flow distribution and influent pump operation. At the Lions Gate plant, the hydraulic factors (flow distribution, SOR and HRT etc.) and wastewater characteristics (settleable TSS and organic content distribution etc.) are considered to be the most important factors affecting the PST performance.

7.5.2 Primary Sedimentation Tank (PST) Performance

Because of the highly variable operating conditions of the primary settling tanks, assessing the performance can only be done on a statistical basis while compliance is measured on a per instance basis using flow proportional composite daily data. Therefore only the extreme values of BOD and TSS are relevant to the understanding of the issue. Achieving compliance of a primary treatment plant under all conditions requires a very substantial factor of safety. Compliance can only be assessed in terms of the likely frequency of failure. Because the relationship of the measurable parameters, such as SOR and HRT, are not well correlated to performance, it is not possible to define a single value which can indicate when a plant requires upgrading. At best a trend can be indicated and the probability of exceedance can be associated with that trend. This is the approach, which has been adopted here.

Figures 7.4 and 7.5 illustrate the distributions of effluent TSS and BOD concentrations (flow-proportional daily composites) respectively for the period of 1991 to 2003. Historical effluent TSS concentration complied comfortably with the Permit limits, whereas effluent BOD concentrations showed some exceedances above the limit in the year 1993, 1996, 1998 and 1999.

FIGURE 7.4
EFFLUENT TSS CONCENTRATION AT LGWWTP (1991-2003)

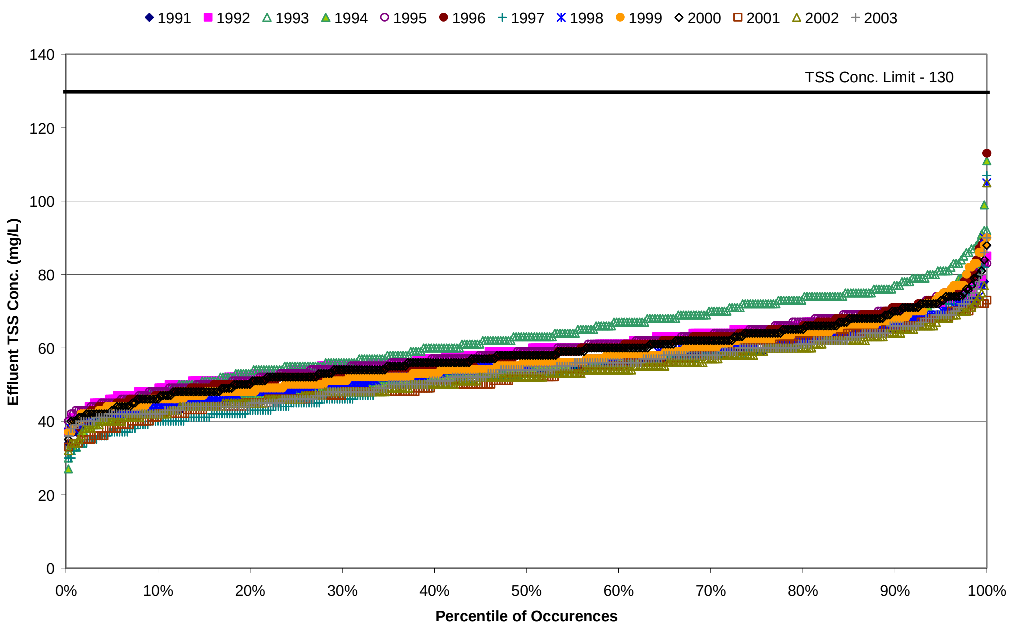
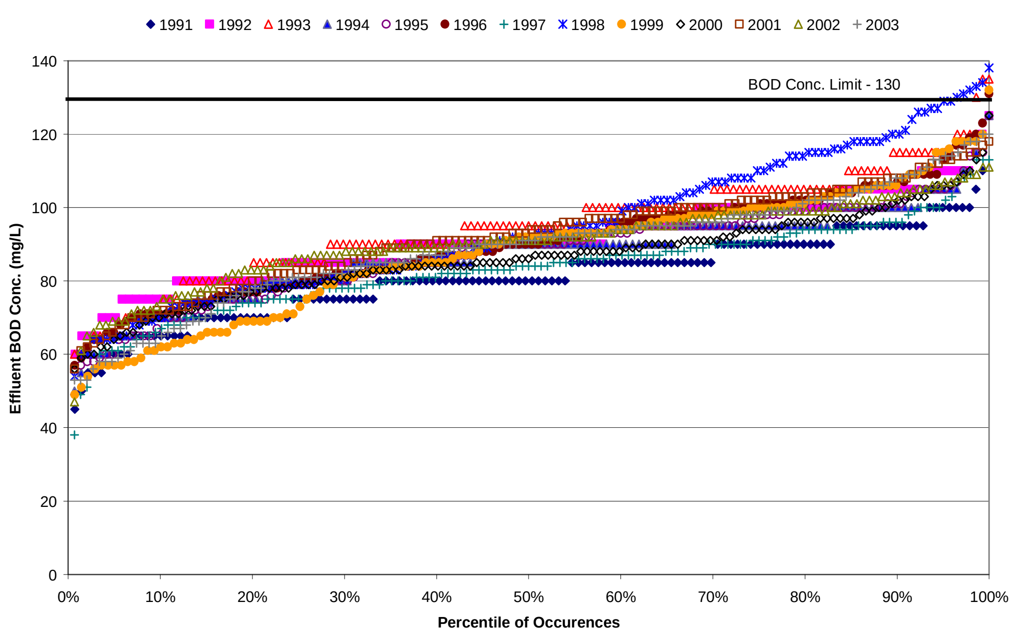


FIGURE 7.5
EFFLUENT BOD CONCENTRATION AT LGWWTP (1991-2003)



The variability of the data and the correlation between SOR and % removal of TSS and BOD for the years 2001 and 2002 are assessed. The trend to poorer removal with increasing average day SOR is apparent. Further discussion can be found in Section 3 of Appendix 10.

7.5.3 Forecast Effluent Quality

Figures 7.6 and 7.7 illustrate the effluent quality for TSS and BOD respectively in percentile of occurrences for LGWWTP. The methodology and assumptions used in developing these graphs is discussed in Section 3 of Appendix 10. Flow and load for the design case are used in establishing projections for year 2004 and 2031.

The forecast of the probable level of compliance is sensitive to the assumption made. More detail is provided in Appendix 10.

FIGURE 7.6
LGWWTP PROJECTED EFFLUENT TSS CONCENTRATION IN PERCENTILE OF OCCURRENCE

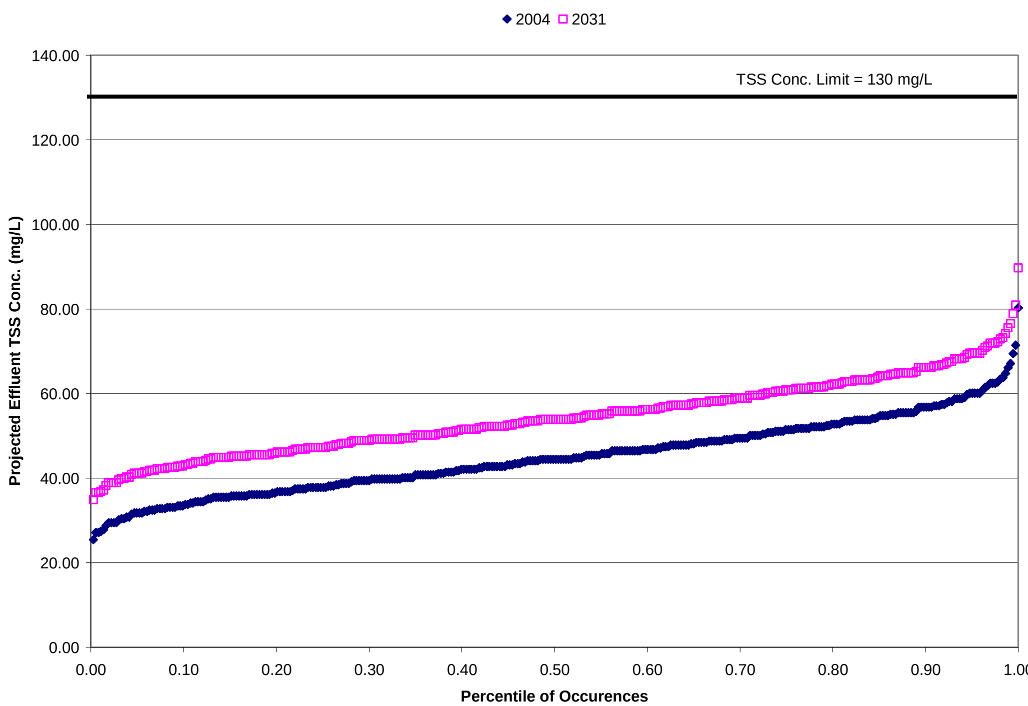
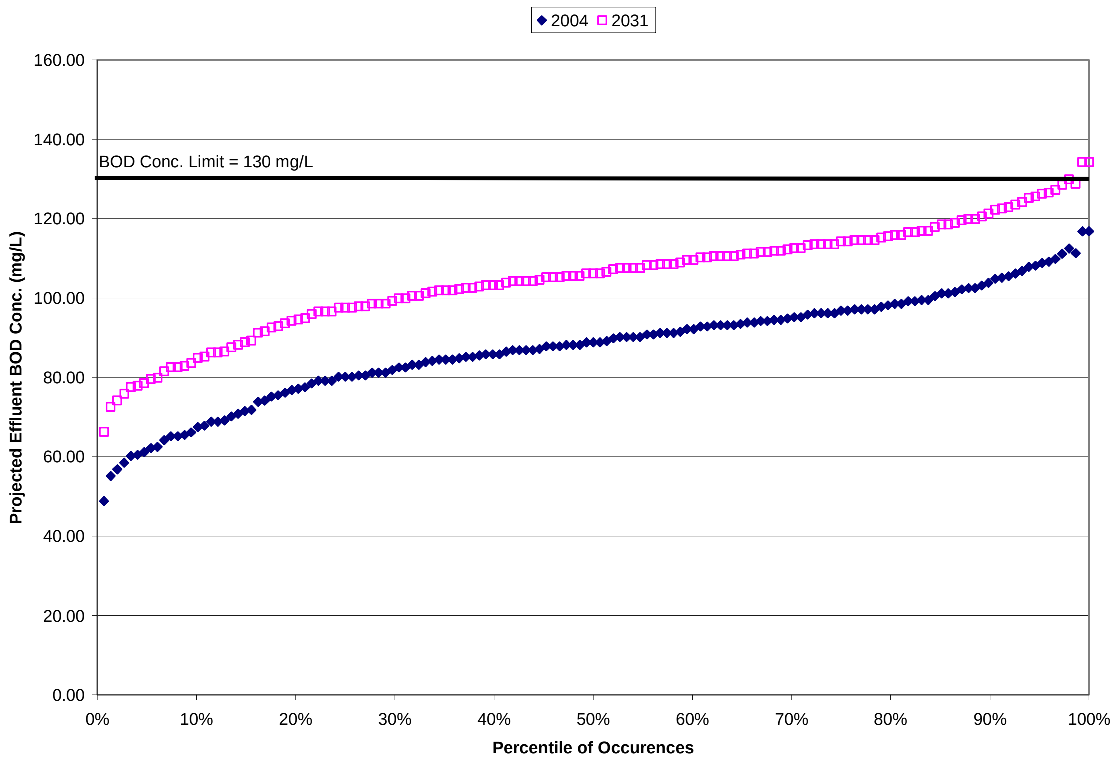


FIGURE 7.7
LGWWTP PROJECTED EFFLUENT BOD CONCENTRATION IN PERCENTILE OF OCCURRENCE



8.0 SUMMARY OF TECHNICAL STUDIES ON SPECIFIC ISSUES

8.1 SMALL SCALE TESTING

Previous study has identified anionic surfactants, which are measured as methylene blue active substances (MBAS), as the primary cause of toxicity at the Lions Gate WWTP. Limited sampling and analysis indicates that the influent MBAS concentration at Lions Gate is typically about 2-4 mg/L from 8 AM until late morning, and then increases to a peak as high as 10 mg/L to 11 mg/L from about 4 PM to midnight.

The pilot-testing program was designed to conduct parallel tests on samples of settled sewage leaving the primary settling tanks. The purpose of the parallel tests was to compare the effectiveness of chemically enhanced primary treatment (CEP) with that of partial biological treatment, and also with that of CEP followed by partial biological treatment, in reducing the acute toxicity of the effluent at Lions Gate (acute toxicity as measured by the 96-hour LC_{50} rainbow trout bioassay). Evaluation of partial biological treatment was undertaken using biological waste sludge taken from the Annacis Island WWTP. Each batch test was done in parallel onsite at Lions Gate, using settled sewage from that facility, combined with waste biological sludge from Annacis.

Comparisons among the various treatments should be taken as subjective; that is, since parallel tests were conducted on the same sample of settled sewage each time, relative comparisons regarding the effectiveness of one treatment compared to the others are valid. However, the results should not be projected to full-scale WWTP performance.

The results of the acute toxicity bioassay testing at Lions Gate (96 hr LC_{50}) are summarized in Table 8.1.

TABLE 8.1
LIONS GATE WWTP TOXICITY RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Fail
CEP	Fail	Pass	Fail	Pass	Fail	Fail
25% Biological	Fail	Fail	Fail	Fail	Fail	Fail
50% Biological	Fail	Pass	Pass	Fail	Fail	Fail
CEP+25% Biological	Pass	Pass	Pass	Fail	Pass	Pass

As shown in Table 8.1, the control (untreated) effluent samples consistently failed the bioassay at Lions Gate. Twenty five percent biological treatment was relatively ineffective in improving removal of TSS, TBOD, and SBOD compared to the other treatment processes, and was similarly ineffective in improving toxicity test results.

The samples of primary effluent from the Lions Gate WWTP contained material that exerted a high oxygen demand in four of the six batch tests. Oxygen starvation was the most probable cause of the observed 100% lethality within the first two hours in the control samples in these four tests.

The results of this study did not support the MBAS toxicity threshold of 2 mg/L to 2.5 mg/L identified by others. One hundred percent survival was observed in several treated samples that contained MBAS concentrations in the range 4 mg/L to 6 mg/L, and 90% survival was observed in one sample that contained 7.8 mg/L MBAS. Fish mortality in the treated samples from the batch tests was not directly related to MBAS concentration. In three of the six batch tests, samples that were found to be non-acutely toxic according to the bioassay had higher MBAS concentrations than samples that were acutely toxic within the same batch test. The concentration of MBAS is therefore not a reliable indicator of fish toxicity, nor does the greatest degree of MBAS removal appear to result in the greatest improvement in toxicity testing results.

Chemically enhanced primary treatment (CEP) followed by 25% biological treatment was the most effective of all treatments tested in improving toxicity test results at Lions Gate (83% improvement compared to the control), followed by 50% biological and CEP alone at 33% improvement.

Additional study is needed to evaluate the toxicity of individual anionic surfactants contained in the influent to the Lions Gate WWTP, and the removal rates of those surfactants by chemical and biological processes (this is currently being undertaken by UBC Civil Engineering) and by GVRD.

8.2 PLANT CONDITION

8.2.1 Introduction

A site visit was carried out in 2003 to examine the current condition of the treatment equipment and unit processes. The purpose was to determine how the current primary treatment plant could be integrated with the proposed secondary treatment processes.

8.2.2 Inlet Screens and Pump Station

The existing two screens have a capacity that will almost meet a potential increase of the peak wet weather flow of 356 Ml/d provided a third similar screen is installed in the available space to provide adequate redundancy. However problems of grit accumulation and screw conveyor overload will remain.

The existing pump station does not currently have adequate redundancy at peak wet weather flow. It could be upgraded for redundancy by installing one pump equal to the existing largest units and to meet any increase in peak wet weather flow. Alternatively larger pumps could be utilized to provide the required redundancy. Any amendment of the entry arrangements for the pumps would be difficult to implement since the suction piping is encased in the foundation of the caisson structure.

A detailed study is recommended to identify upgrade options.

Considering that build-out to secondary will not be carried out until 2030, the existing pump station and screens will be over 50 years old at that time. A case could be made that this system will have reached the end of its lifespan before 2030 and should be replaced.

8.2.3 Grit Removal

The existing aerated grit removal tanks do not perform well and as a result grit causes problems in the digesters and results in premature wear on the centrifuges. Considering that build-out to secondary will not be carried out until 2030, the existing grit removal system will be over 70 years old at that time. A case could be made that this system will have reached the end of its lifespan by 2030 and should be replaced.

8.2.4 Primary Clarifier

A required capacity increase could be achieved by the demolition of end walls and the extension of several of the primary clarifier tanks. The construction of side walls and end walls with launders, as well as the extension of the sludge and scum scraper mechanisms would be required.

8.2.5 Disinfection and Dechlorination

The existing chlorine contact tanks could remain in service until 2031. Thereafter a UV disinfection system could be considered in order to minimize potential environmental risks associated with chlorination by-products.

8.2.6 Sludge Thickening

The existing tank is currently used for sludge storage and allows for the semi-batch operation of the anaerobic digester. A second gravity thickener should be installed to provide redundancy and the capacity needed for increasing loads prior to 2031.

Should partial secondary treatment be installed, it is proposed to thicken the waste activated sludge with DAF sludge thickeners.

8.2.7 Digesters

Digesters 1 and 2 should be demolished to make space for upgrading the plant headworks, or chemical dosing systems.

8.2.8 Dewatering Systems

The required capacity for the future, to at least 2046, can be achieved by installing an additional centrifuge in the space provided and increasing the operating times of the centrifuges.

8.2.9 Odour Control

The existing odour control facility on the sludge dewatering building will be retained. The need and design criteria for upgrading of odour control facilities on the plant should be investigated. Significant odour control up grades may be required.

8.2.10 Effluent Outfall and Diffusers

The capacity of the existing system may be inadequate under certain combinations of high sewage flows and high tides. A study is recommended to define the probability of failure.

8.3 GEOTECHNICAL CONSIDERATIONS

8.3.1 General

A preliminary geotechnical assessment was carried out for the Lions Gate Wastewater Treatment Plant. The assessment included a review of the subsoil conditions, foundations, seismic ability and potential rise in sea level.

A preliminary assessment and recommendations regarding the proposed pipeline routing from the Lions Gate Wastewater Treatment Plant to the Iona Island Wastewater Treatment Plant are provided in Section 13.

A brief summary of the assessment and recommendations are given below.

8.3.2 Detailed Assessment

Subsoils at the site consist of a maximum 1.8 m of fill comprising sand and gravel with pieces of wood, debris and organics; a 13 to 15 m thick layer of sand and gravel with some cobbles and boulders; a 25 to 35 m thick layer of silty sand with some gravel; a 20 to 40 m thick very dense glacial till overlying claystone bedrock.

For the design 1:475 year return period earthquake motion, potential liquefiable zones at this location are expected to be scattered throughout the site, with some local zones of significant liquefaction. Earthquake shaking together with subsoil liquefaction would likely cause ground settlement and movement towards Burrard Inlet. Ground improvement to prevent lateral spreading is recommended together with foundation upgrades such as soil anchors, minipiles, and steel pipe piles. The cost of providing ground densification for lateral support is estimated at $1,680,000 and the cost of soil anchors for the existing structures is estimated at $1,480,000. These amounts are included in the capital cost estimates for interim and build-out to secondary.

Seismic design parameters and lateral earth pressure on basement walls are provided for soil-structure interaction analyses.

The assessment results given are provided for planning purposes only. Detailed design and analysis are needed for the final design. The detailed analysis would require subsoil data, which would have to be obtained from site-specific drilling methods such as Cone Penetration Tests, Standard Penetration Tests and/or other equivalent methods. The analysis would include liquefaction assessment, estimation of seismically induced ground deformation, foundation bearing capacity and settlement measurements.

8.4 SEISMIC CONSIDERATIONS

8.4.1 General

This section provides an overview of seismic considerations of the structures at the Lion's Gate Wastewater Treatment Plant.

The assessment involved a review of the existing drawings of the structures and applying the current National Building Code of Canada to analyze the structures for a 1 in 475 year return period design basis earthquake.

Many of the existing structures were designed and built between 1960-1980 and are classified as "Post Disaster Buildings". Before 1980, code requirements to design water retaining structures for earthquake conditions were less stringent than the current National Building Code of 1995 and the British Columbia Building Code of 1998. Formulas specified by National Building Codes to design minimum lateral seismic force have two basic factors, which have significant effects on the results. These are the Importance Factor and the Foundation Factor, designated I and F respectively. Before 1980 the factors for post disaster buildings were I = 1.3, and F = 1.3. In recent Codes they are now I = 1.5, and F = 1.5 to 2.0. Many of the structures in the plant have therefore been checked and analyzed for about 30% to 75% more loads than they were originally designed for.

The following assumptions have been made:

1. After a 1:475 year design earthquake event:

    a) The tanks must remain usable. Slight structural damage is allowable and insignificant leakage can occur.

    b) The tanks must remain usable, but may suffer repairable structural damages and can be taken out of service, then inspected and repaired in a reasonable time.

2. Steel & concrete strength, ground acceleration and velocity

    a) Before 1980
    Design steel strength was 40 ksi (280 MPa)
    Design conc. strength was 3000 psi (21 MPa)

    After 1980
    Design steel strength is 60 ksi (420 MPa)
    Design conc. strength is 4200 psi (30 MPa)

    b) Before 1980
    Ground acceleration and velocity = 0.20 g.

    After 1980
    Ground acceleration and velocity = 0.2 – 0.5 g

3. Capacity/Demand Ratio (C/D)

   A capacity/demand ratio less than unity indicates that the structure is inadequate or overloaded.

8.4.2 Conclusions

- Digesters 1 and 2 will not survive the design earthquake.
- Digester 3 should be checked in more detail to confirm that some reported upgrade improvements have occurred.
- Digester 4 is marginal in its ability to resist the design earthquake.
- There are 3 ton and 3.5 ton precast panels around Digesters #3 and #4 resting on a ring shaped foundation slab and anchored to digester walkways at the top using steel angles and bolt connectors. Vertical and horizontal movements due to post liquefaction could fail the rigid connections at the top of the panels and welded connections between the panels and may cause them to collapse.
- The pre-aeration and sedimentation tank roofs should be upgraded.
- Unless liquefaction mitigation measures are implemented, most of the expansion joints could become damaged. Vertical movements can rupture PVC waterstops in joints or destroy the bond between the waterstops and the concrete, especially the expansion joint in the effluent channels between Stage I and Stage II tanks. There could be significant damage unless some mitigating action is taken.
- Post liquefaction movements could fracture the connections in all pipelines, especially the 750 mm ø and 600 mm ø pipelines between the pump station and pre-aeration tanks.

8.5 ALTERNATIVE SITE FOR NEW PLANT

Introduction

It would be feasible to construct a new treatment plant east of the existing Lions Gate WWTP. LGWWTP, with the exception of the influent pump station, could then be demolished and the site made available for other uses.

Wastewater would be collected at the existing LGWWTP pump station and pumped through a new pipeline to a new treatment plant site. Most of the wastewater originating in North Vancouver would gravitate to this proposed plant. Treated effluent would be pumped back from the new plant to the LGWWTP site where it would be discharged to the Burrard Inlet through the existing outfall. Flows and plant sizing are based on a 2046 design horizon. The location of an alternative site is assumed to be within 1 km of the existing plant.

Forcemains

An allowance has been made for the construction of two new forcemains, one to convey untreated wastewater from the existing LGWWTP pump station to a new WWTP on another site, and the other to convey treated wastewater from the proposed treatment plant to the existing outfall.

Pump Stations

The estimated 100 ML/d PWWF from LGWWTP to an alternate WWTP site is a fraction of the existing influent pump capacity. No allowance has therefore been made for pump station improvements.

The new effluent pump station would include pumps capable of pumping 420 ML/d at a TDH of 9 m. Based on Stantec and D&K cost curves, the estimated construction cost is $2.8 million. Allowing 45% for redundancy, engineering and contingencies, the estimated total cost is $4 million.

Treatment Plant

The new WWTP would be constructed on a site within one 1 km of the existing plant. The estimated land area is 6.9 Ha. Preliminary cost estimates for the total project are detailed in Table 8.2.

Construction cost estimates are based on D&K Cost Curves. Total Costs are estimated to be Construction Costs x 1.4 and are inclusive of additional items such as noise control, earthquake protection, odour control, architectural finishes, outfall, contingencies, engineering, financing and administration. Estimates are based on an ENR Index of 6794 (November 2003).

TABLE 8.2
COST ESTIMATES – ALTERNATIVE SITE FOR NEW PLANT

Description	Flow (ML/d)	Area (Ha)	Construction Cost $\$10^6$	Total Cost $\$10^6$
New WWTP	131 (ADWF)	6.9	109	153
New Pump Station	420 (PWWF)	Included	3	4
Forcemains	-	-	2	3
Totals	-	**6.9**	**114**	**160**

8.6 ALTERNATIVE SITES FOR MULTIPLE PLANTS

Introduction

Real estate available for the future development of LGWWTP is limited. An alternative approach could consider treatment at three dispersed sites, the existing LGWWTP and at two other plants. The costs and benefits of this strategy are briefly reviewed in this section.

Plant sizing is based on projected flows for the year 2046. Basic descriptions of each plant are provided below.

West Vancouver Waste Water Treatment Plant

The design ADWF would be 26 ML/d. The plant would be located in the vicinity of Ambleside Park. Treated wastewater would be discharged into Burrard Inlet through a new outfall.

Lions Gate Waste Water Treatment Plant

This plant would be located at the existing LGWWTP site and would use the existing outfall and infrastructure on the treatment plant site. Secondary treatment would be designed for 66 ML/d ADWF.

Lynn Pump Station Waste Water Treatment Plant

This plant would be located in an industrial zone near the existing Lynn Pump Station and designed for 39 ML/d ADWF. Discharge would be into Burrard Inlet upstream of the Lions Gate Bridge and may require biological nutrient removal.

Credit for Existing Sewers

As the wastewater would be distributed to three treatment plants, it would not be necessary to upgrade some North Shore trunk sewers that would have to be upgraded if all flows were directed to LGWWTP or to a single replacement site. A credit of $5 million has been allowed for twinning the North Vancouver City Section trunk sewer. For estimating purposes this has been assumed to be a 915 mm (36 in.) diameter sewer with a length of 7.5 km.

Construction cost estimates are based on D&K cost data. Total Costs are estimated to be Construction Costs x 1.4 and are inclusive of additional items such as noise control, earthquake protection, odour control, architectural finishes, outfall, contingencies, engineering, financing and administration (Table 8.3). Estimates are based on an ENR Index of 6794 (November 2003).

TABLE 8.3
COST ESTIMATES – ALTERNATIVE SITES FOR MULTIPLE PLANTS

Treatment Plant	ADWF (ML/d)	Area (Ha)	Construction Cost $\$10^6$	Total Cost $\$10^6$
West Vancouver inc. outfall	26	1.8	38	53
Lions Gate Deduction for existing infrastructure	66	3.4	66* (27)	92* (38)
Lynn P/S inc. outfall	39	2.9	56	78
Totals	**131**	**8.1**	**133**	**185**

*: Greenfield construction cost

Excluding land cost, the total project cost of a single new 131 ML/d plant near the existing plant is estimated to be $160 million while the cost of three dispersed plants is $185 million. The premium on the capital cost for dispersed treatment would therefore be approximately 16%.

O&M costs for dispersed treatment would be higher than for a single treatment plant. The cost of power and chemicals would be approximately equal. However, additional manpower resources would be required, particularly as the Lynn P/S plant could be a BNR plant, which would require a higher level of control. Monitoring costs for the three plants would be higher. Annual plant maintenance costs would also be higher.

<u>Sludge Treatment</u>

Lynn Pump Station WWTP and West Vancouver WWTP would probably not include sludge digestion facilities, as use would be made of the digesters at the Lions Gate WWTP. Sludge would be conveyed to the plant using existing sewers.

<u>Discussion of Treatment at Multiple Plants</u>

Given the existence of a trunk sewer system delivering to the LGWWTP site, the creation of a dispersed secondary treatment system has little advantage to offer. The following disadvantages have been identified:

- Difficulty of acquiring land
- Higher project cost
- Higher operating and maintenance cost
- More monitoring and administration
- Increased social impact

9.0 SELECTION OF PREFERRED TREATMENT – BUILD-OUT TO SECONDARY

9.1 SUMMARY OF ALTERNATIVES CONSIDERED

An extensive review of secondary treatment processes was undertaken in order to evaluate the most effective and affordable options for the IIWWTP and LGWWTP. A summary of the process considered is included in Section 4.1 and will not be repeated in this section.

9.2 RESULTS OF FIRST LEVEL OF SCREENING

An initial screening process was utilized to determine the most feasible options for the build-out to secondary process. A complete description of the method is presented in Section 8 of Appendix 3. A summary is included in Section 10.2 of this report. The initial screening resulted in the following options being selected for the build-out to secondary process:

- Option 1: Primary + 100% of 2 x ADWF TF/SC
- Option 2: Primary + 100% of 2 x ADWF BAF
- Option 3: 2 x ADWF HRAS + Primary
- Option 4: CEP + 60% of 2 x ADWF TF/SC
- Option 5: 100% of 2 x ADWF TF/SC in parallel with Primary

9.3 SUMMARY OF OPTIONS THAT PASSED FIRST LEVEL OF SCREENING

Typical design values were used to develop a comparison between the five options which passed the first level of screening. This approach provided a consistent comparison level for each of the 5 options. Schematic process flow diagrams, conceptual plant layouts, footprint requirements, sludge production, effluent quality projections, capital and O&M cost estimates, process flexibility and other factors were developed and evaluated. The following is a brief summary of that assessment.

9.3.1 Option 1 - Trickling Filter/Solids Contact (TF/SC)

The primary treatment process will continue to treat 100% of the PWWF. The TF/SC process will be designed to treat 2 x ADWF (262 ML/d) at 100% Maximum Month Load (MML) (19.5 t/d BOD and 18 t/d TSS) following primary treatment. Final clarifiers will be used to remove TSS and biosolids generated from the TF/SC process. In order to avoid the environmental risk associated with chlorinated organic compounds, it is assumed that secondary treated effluent will be disinfected using UV. Flows greater than 2 times ADWF will bypass secondary treatment and be discharged directly to the chlorination system and the outfall.

Primary sludge will be thickened in gravity thickeners. Secondary sludge will be thickened in DAF thickeners. The thickened sludge will be stabilized in digesters. Digested biosolids will be dewatered. Sludge handling effluents will be recycled.

The additional footprint required for this option is 10,400 m^3. 100% of the available real estate will be utilized. The estimated capital cost is $97 million.

9.3.2 Option 2 - Biological Aerated Filter (BAF)

Similar to Option 1 except that BAF will be used to provide secondary treatment. Flows and loads are as detailed for Option 1. The BAF system does not require final clarifiers.

The additional footprint required for this Option is 4,650 m^2. 45% of the available real estate will be utilized. The estimated capital cost is $93 million.

9.3.3 Option 3 - High Rate Activated Sludge (HRAS)

HRAS will be used in conjunction with primary treatment. Following preliminary screenings, 2 x ADWF will pass through the grit removal facility. This flow (262 ML/d) will then pass to the HRAS process for treatment. Flows greater than 2 x ADWF will receive primary treatment only. Flows from the primary and secondary treatment systems will be handled as described for Option 1. Arrangements for solids handling and sludge treatment effluent are as described for Option 1.

The additional footprint required for this option is 14,120 m^3. About 135% of the available real estate would be required. The estimated capital cost for this option was not developed as the plant will not fit on the site.

9.3.4 Option 4 - Chemically Enhanced Primary (CEP) and 60% TF/SC

CEP will be used to improve the efficiency of the primary system. Following primary treatment 60% of 2 x ADWF (157 ML/d) and 60% MML after primary (13.5 t/d BOD and 7.2 t/d TSS) will be selected as the TF/SC design capacity. Primary and chemical sludge will be thickened in gravity thickeners. Secondary sludge will be thickened in DAF thickeners. Flows from the primary and secondary treatment systems will be handled as described for Option 1. Arrangements for solids handling and sludge treatment effluent are as described for Option 1.

The additional footprint required for this option is 8,400 m^2. 81% of the available real estate will be utilized. The estimated capital cost is $90 million.

9.3.5 Option 5 - TF/FC in Parallel with Primary

This option was developed when it was established that Option 3 (HRAS) would not fit on the site.

TF/SC will be designed to treat 2 x ADWF (262 ML/d) of fine screened sewage and the associated MML (30 t/d BOD and 36 t/d TSS). Final clarifiers will be used to remove TSS and biosolids. In order to avoid the environmental risk associated with chlorination, it is assumed that secondary treated effluent will be disinfected using UV. Flows greater than 2 x ADWF will, after primary treatment, be discharged directly to the chlorination system and the outfall. Arrangements for solids handling and sludge treatment effluent are as for Option 1.

The additional footprint required for this option is 9,100 m^2. 87% of the available real estate will be utilized. The estimated capital cost is $108 million.

9.4 RESULTS OF SECOND LEVEL OF SCREENING

Details of the screening process are described in Section 10.4 of Appendix 3 and summarized in Section 4.4 of this report.

The option selected for the build out to secondary treatment for the Lions Gate WWTP is:

- Option 2: BAF

Results of the screening process are summarized in the following Table 9.1.

TABLE 9.1
SUMMARY OF SECOND LEVEL OF SCREENING

	Option 1		Option 2		Option 3		Option 4	
	TF/SC		BAF		CEP + 60% TF/SC		Primary + TF/SC	
Cost + Tech @ 50%	83.5	2	89.1	1	78.0	4	83.2	3
Environmental @ 50%	89.0	1	88.9	2	81.1	4	88.6	3
Social @ 50%	81.3	2	90.1	1	75.2	4	80.6	3
Rank	2		1		4		3	

In each case the points for all category weightings are summarized to give a total. The ranks shown are based on the total number of points.

9.5 PREFERRED PROCESS OPTION FOR BUILD-OUT TO SECONDARY

The selection of the final recommended process is addressed in detail in Section 10 of Appendix 3 and Section 9 of Appendix 4. The selected process is BAF based on the small footprint, which allows expansion beyond the required capacity forecast for 2046. None of the other options offer this advantage.

The headworks of the plant have not been addressed in detail as part of this study. However it is assumed that since the existing headworks will have reached the end of its lifespan by the time the build-out to secondary is carried out in 2030 new headworks will likely be needed. The sum of $14 million has been included in capital cost estimates for new headworks. If following more detailed analysis, it is determined that complete new headworks are not necessary, this amount could be adjusted.

9.6 GENERAL DESCRIPTION OF PROPOSED BAF PLANT

A layout of the proposed plant is shown on the drawings provided under separate cover (Volume 5, Interim and Build-out to Secondary Stage, Preliminary Design Drawings).

The existing PSTs will be retained without upgrading. Following these, a flow of twice the average dry weather flow (ADWF) of 111 Ml/d = 222 Ml/d is pumped into a BAF system. Flows in excess of 2 x ADWF discharge from the PSTs to the chlorine contact tank and are then blended with the secondary effluent.

The BAF system consists of 10 modules complete with necessary clean effluent backwash water storage tank, dirty backwash water storage tank, aeration blowers for process air, backwash pumps and air scour blowers. The design load on the BAF plant (MML) is 20.4 T/d BOD and 8.8 T/d of TSS. The BOD load limits the treatment capacity of the plant. Back washing is triggered by the build-up of biomass in the filter resulting in increased head losses. The frequency with which back washing can be carried out determines the capacity.

Backwash water from the BAF is treated in a dissolved air flotation system with the effluent discharged to the BAF influent stream (as an alternative, a thickening centrifuge system should be considered).

Primary sludge from the PSTs is thickened in gravity thickeners. Additional thickener capacity is required.

The sludge is treated in thermophilic anaerobic digesters and de-watered in the present centrifuge plant which will be upgraded.

Effluent from the BAF can be treated by UV disinfection or by chlorination depending on which is preferred. The PST effluent would be chlorinated and discharged to the chlorine contact tanks, which would be retained.

9.7 SUMMARY OF ESTIMATED COST OF PROPOSED BAF PLANT

The estimated capital cost of the proposed BAF plant is shown in Table 9.2. Detailed breakdowns of the cost estimates are included in Appendix B of Appendix 10. All capital cost estimates are expressed in November 2003 dollars.

The capital costs include the following:

- Seismic upgrading of the existing site by providing ground improvement in a berm along the shore.
- Soil anchors to reduce the probability of flotation of existing structures.
- Engineering (16%) project management/quality control (4%), contingency (30%) and GST (0%).

The following capital costs are excluded:

- Upgrading of the inlet pump station
- Upgrading of the inlet screens
- Upgrading of grit removal facilities

TABLE 9.2
LGWWTP CAPITAL COST ESTIMATE FOR BUILD-OUT OPTION

YEAR	BUILD-OUT
Option	2046
	2 x ADWF BAF
CAPITAL COSTS	
Site Improvements	$4,466,127
Chemical Dosing	$0
Primary Clarifiers	$0
Bioreactor	$23,649,647
Gravity Thickeners	$663,351
DAF Thickeners	$7,458,941
Digesters	$8,885,222
Mechanical Dewatering	$1,254,277
UV	$2,220,000
Odour Control System	$500,000
Site Works	$3,667,800
Admin/Maint. Building	$2,000,000
Control System	$1,785,258
Electrical Substation (allow)	$115,000
Existing Facility Upgrades	$14,200,000
Sub - Total	$70,865,623
Division 1 Cost	$1,659,987
Engineering	$11,338,500
Project Management/QA/QC	$2,834,625
Contingency	$21,259,687
Total Capital Cost	$107,959,000

The estimated operating and maintenance cost (November 2003 dollars) for build-out to secondary at 2046 flows is shown in Table 9.3. The existing primary plant has a staff of 12. For the interim upgrade it is estimated that the staff would increase to 14 persons for Option 1 and to 15 persons for Options 2A and 2B. For the build-out to secondary it is estimated that the staff would increase to 16 persons. Maintenance costs are estimated at the existing cost plus a fixed 0.80% of the capital cost.

The residual management costs are estimated based on a rate of $100/tonne for hauling, reuse (e.g. land application), and other fixed expenses, assuming that land application sites are available.

TABLE 9.3
LGWWTP ANNUAL OPERATING & MAINTENANCE COST ESTIMATE FOR BUILD-OUT OPTION

YEAR	BUILD-OUT
	2046
Option	2 x ADWF BAF
O&M COSTS	
Labour	$1,859,000
Chemical Costs	$140,000
Biolite replenishment	$50,000
Residuals Management	$2,083,000
Energy	$863,000
Repair/Maintenance	$2,194,000
Administration and others	$721,000
Land and building Lease	$332,000
Total (O&M Costs)*	$8,242,000
Total (O&M Costs)**	$4,094,000

The life cycle cost (LCC) of the build-out option is included in Table 9.4. The LCC are based on the following parameters:

- Discount rate: 6% p.a.
- Base date for costing: November 2003
- Evaluation period for build-out (O&M): 2031 to 2060
- Construction period for build-out: 2028 to 2030 (1/3 of capital each year)
- O&M cost basis: Upgrade net O&M cost

TABLE 9.4
LGWWTP LIFE CYCLE COST FOR BUILD-OUT OPTION

STAGE	INTERIM			BUILD-OUT
YEAR	2031	2031	2031	2046
OPTION	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Discounted O&M Cost	$13,095,343	$11,910,007	$18,773,623	$11,685,852
Discounted Capital Costs	$12,437,680	$23,923,074	$27,318,373	$23,757,336
Total Discounted Capital and O & M Costs at present value	$25,534,000	$35,834,000	$46,092,000	$35,444,000

10.0 SELECTION OF PREFERRED TREATMENT OPTIONS – INTERIM TREATMENT

10.1 SUMMARY OF ALTERNATIVES CONSIDERED

A review of thirteen different treatment processes was conducted as part of the selection of the preferred options for the interim upgrade for the LGWWTP. The options range from exclusively chemical/physical process to complete biological treatment. Several options involved a combination of chemical, physical and biological treatment. All thirteen options were evaluated and rated prior to receiving a first level of screening. The options that passed the first level of screening where evaluated in more detail before selecting the short list of preferred options. All the options that were considered are briefly reviewed in this section. A summary of the processes considered is included in Section 5.1 and will not be repeated in this section. Detailed descriptions are presented in Appendix 3.

10.2 RESULTS OF FIRST LEVEL OF SCREENING

The first level of screening involved an initial pass/fail analysis for each of the options based on the following criteria:

- Proven technology
- Discharge requirements
- Reliability
- Site suitability

Options that did not pass each criteria were assessed to determine if the results would change due to technological improvements. If so, they were carried over to the evaluation stage. Options which passed the initial screening were evaluated using a "Delphi" ranking exercise. The Delphi process is described in Section 8.1 of Appendix 3. The "Delphi" exercise allowed a panel of experts to rank the various options based on a set of criteria, including:

- Capital cost
- Operating cost
- Reliability
- Integration
- Flexibility
- Environmental
- Social

The initially ranked options were further evaluated based on a series of factors: incorporation of the existing primary facilities, need to expand to the ultimate build-out scenario, the extent of the interim treatment requirements and the limited need for nutrient removal. The final ranking and options to be analyzed in detail were:

1. Primary followed by 50% ADWF Biological Aerated Filter

2A. 50% ADWF Roughing Trickling Filter in parallel with Primary Treatment

2B. 100% ADWF Roughing Trickling Filter in parallel with Primary Treatment

3. Chemically Enhanced Primary followed by 50% ADWF Roughing Trickling Filter

4. 50% ADWF High Rate Activated Sludge in parallel with Primary Treatment

10.3 ANALYSIS OF OPTIONS THAT PASSED THE FIRST LEVEL OF SCREENING

Typical design values were used to develop a comparison between the five options which passed the first level of screening. This approach provided a consistent comparison level for each of the 5 options. Schematic process flow diagrams, conceptual plant layouts, footprint requirements, sludge production, effluent quality projections, capital and O&M cost estimates, process flexibility and other factors were developed and evaluated. The following is a brief summary of that assessment.

Option 1

The primary treatment process will continue to treat 100% of the PWWF. The BAF process will be designed to treat 50% ADWF (58 ML/d) at 50% MML (8.45 t/d BOD and 7.75 g/d TSS) following primary treatment. The BAF process does not require final clarifiers. After primary treatment, flows greater than 50% ADWF will bypass secondary treatment and discharge directly to the chlorination system and outfall. The estimated capital cost is $55 million.

Option 2A

50% of the ADWF will be diverted to the RTF with measures being taken to minimize the amount of grit diverted. The balance of the flow will flow to the existing headworks and primary system and from there to the chlorination system and outfall. The RTF system is designed for 58 ML/d. This is associated with 50% MML (12.5 t/d BOD and 14 t/d TSS). Final clarifiers will be installed downstream of the RTF before the flow is directed to the chlorination system and the outfall. Combined primary and biological treatment will be thickened in DAF units and digested. The estimated capital cost is $35 million.

Option 2B

This option is similar to Option 2A with the exception that, based on small-scale testing, 100% ADWF will be directed to the RTF which will treat 16 ML/d and 90% MMF loadings after primary treatment (25 t/d BOD and 28 t/d TSS). Costs are higher than Option 2A but effluent quality is significantly improved. The estimated capital cost is $45 million.

<u>Option 3</u>

This option is similar to Option 2A except that primary treatment precedes the RTF and the primary treatment is chemically enhanced. No final clarifiers are provided downstream of the RTF. After the CEP process, 50% ADWF (58 ML/d) will be treated in the RTF process. RTF design loadings are 50% MML (6.5 t/d BOD and 3 t/d TSS). Flows greater than 50% ADWF will bypass the secondary units. Primary and chemical sludge will be thickened in gravity thickeners. The estimated capital cost is $40 million.

<u>Option 4</u>

Following preliminary screening 50% ADWF (58 ML/d) is diverted to HRAS in a way that minimizes the diversion of grit. The remainder will pass through the primary system. Primary and secondary effluents will be combined and discharged to the chlorination system and outfall. There is insufficient space available for this option and a capital cost estimate was therefore not developed.

10.4 RESULTS OF THE SECOND LEVEL OF SCREENING

A second level of screening was under taken to assess the merits of each of the five short listed options. Unit process sizing and configuration was developed for each option based on typical design values and preliminary cost estimates. A points weighted evaluation matrix was used to compare processes, taking into account various elements specific to the LGWWTP site. The three major factors used for the evaluation were, cost and technical, environmental and social. Following the evaluation Option 2A, the 50% RTF, was ranked highest. Option 2B, 100% RTF was the second and Option 1, 50% BAF was assessed as the third ranking option. The difference in the scores for these options was negligible and all must be considered equal.

Considerations, which must be assessed include the practicality of the interim option being expanded to build-out. The BAF option also had the top score in the ranking for the build-out to secondary, making it a preferred option for the interim period.

Details of the screening process are summarized in the following table:

TABLE 10.1
SUMMARY OF SECOND LEVEL OF SCREENING

	Option 1		Option 2A		Option 2B		Option 3	
	50% BAF		50% RTF		100% RTF		CEP + 50% RTF	
Cost + Tech @ 50%	80.3	2	81.9	1	79.8	3	76.2	4
Environmental @ 50%	78.8	4	85.0	1	84.8	2	79.7	3
Social @ 50%	83.1	1	80.0	2	78.3	3	73.7	4
Rank	3		1		2		4	

10.5 PREFERRED PROCESS OPTIONS FOR INTERIM UPGRADES

The selection of unit operations is restricted to those, which are compatible with the Build-out to Secondary. In order to ensure compliance until 2031 either CEP treatment or partial biological treatment is assumed to be required. The use of combined CEP and biological treatment offers further improvement in effluent quality over partial biological treatment. The following options were assessed:

Option 1 - CEP
Option 2A - 50% ADWF BAF
Option 2B - CEP + 50% ADWF BAF

Upgrading of the headworks of the plant is required as described in Section 9.5. The layout of the options is shown on the drawings provided under separate cover (Volume 5, Interim and Build-out to Secondary Stage, Preliminary Design Drawings).

<u>Option 1 – CEP</u>

The existing PSTs are retained without increase in area. The CEP process is sized to provide 70 mg/l of Alum upstream of the PSTs. The required upgrading of the solids handling systems is assessed. This indicates a requirement for an increase in the sludge thickener capacity and in the digester capacity. Centrate discharges to the plant influent stream. Disinfection using chlorine is continued.

<u>Option 2A – 50% ADWF BAF</u>

The existing PSTs are retained without increase in area. Flow downstream of the PSTs is pumped to a BAF with a capacity of 50% of ADWF = 50% x 104 Ml/d = 52 Ml/d. The BAF is sized to treat the load associated with a plant of 50% ADWF capacity. As indicated in Appendix 3 Figure 9.22, the BOD load is 50% of the total load on the plant. In this case the MM loads are 9.0 T/d BOD and 5.5 T/d of TSS. Supporting unit processes required are similar to those required for Build-out to Secondary. Disinfection using chlorine is continued.

<u>Option 2B – CEP plus 50 % ADWF BAF</u>

The plant is configured as for Option 2A with the addition of CEP dosing facilities. The capacity of the BAF has been retained at the same level as for Option 2A so that the implications of partial CEP treatment can be assessed.

10.6 OPTION IMPLEMENTATION SCENARIOS (FOR OPERATIONAL CERTIFICATE COMPLIANCE)

The Liquid Waste Management Plan (LWMP) has indicated that the base level of treatment for Lions Gate WWTP should meet the following maximum daily concentration levels:

- BOD_5 130 mg/L
- TSS 130 mg/L

An analysis has been carried out to assess the projected effluent quality compliance level for each interim option at 2031. Figures 10.1 and 10.2 illustrate the analysis results for effluent TSS and BOD concentration respectively. Figure 10.1 shows that Effluent TSS concentration reliability levels reach 100% for all options even when no upgrade is implemented.

FIGURE 10.1
LGWWTP PROJECTED RELIABILITY LEVEL OF EFFLUENT CONCENTRATION FOR TSS

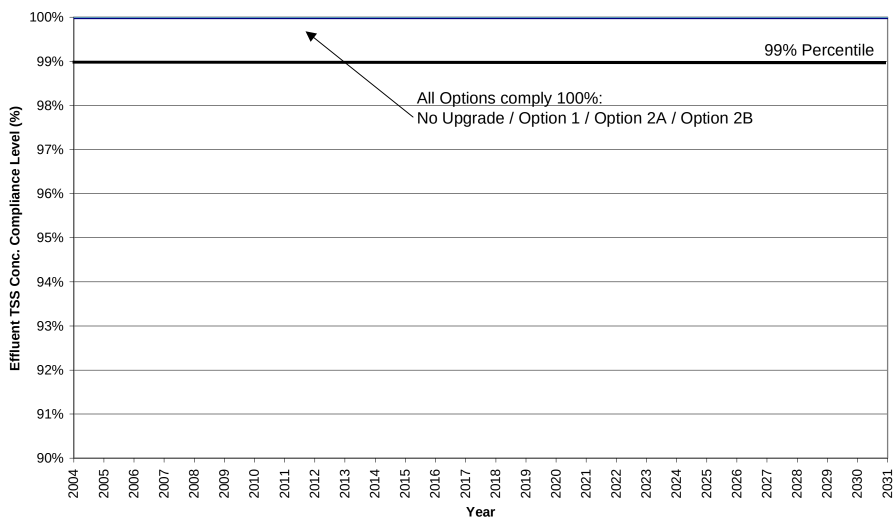


FIGURE 10.2
LGWWTP PROJECTED RELIABILITY LEVEL OF EFFLUENT CONCENTRATION FOR BOD

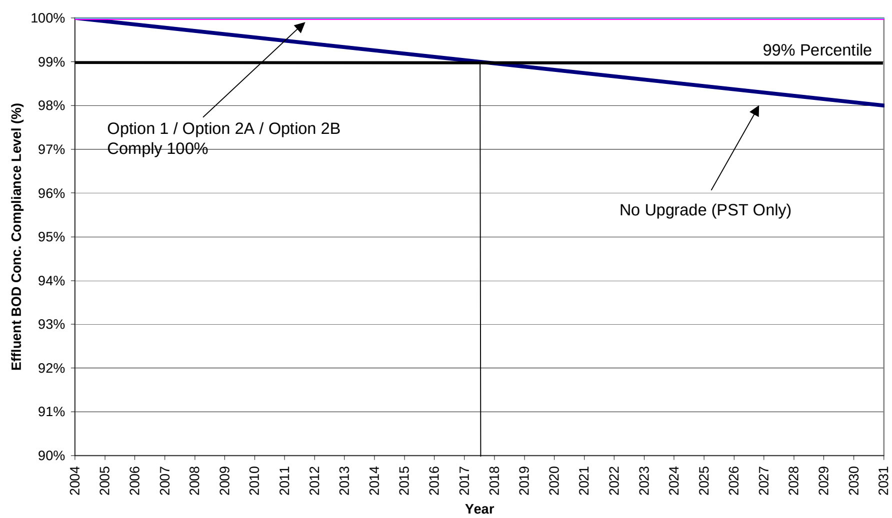


Figure 10.2 shows that BOD concentration is the controlling factor for the upgrade. Without an interim upgrade the effluent BOD concentration reliability level drops to 98% in 2031. This should be compared with the target level of 99% reliability The curves also show that any preferred interim upgrade option can bring the plant to 100% BOD concentration reliability regardless of the projected future increase in flow and load.

10.7 SUMMARY OF ESTIMATED COSTS FOR PROPOSED OPTIONS

The estimated capital costs, operating and maintenance and life cycle costs (LCC) for the preferred interim options are summarized in Table 10.2, 10.3 and 10.4 respectively. The assumptions and parameters used for estimating the costs are discussed in Section 9.7, with the following parameters applicable to interim:

- Evaluation period for interim: 2004 to 2030
- Construction period for interim: 20014 and 2015 (1/2 of capital each year)
- Commissioning date for interim: 2016

TABLE 10.2
LGWWTP CAPITAL COST ESTIMATES FOR INTERIM OPTIONS

YEAR	INTERIM		
Option	2031	2031	2031
	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF
CAPITAL COSTS			
Site Improvements	$4,056,768	$4,056,768	$4,056,768
Chemical Dosing	$500,000	$500,000	$500,000
Primary Clarifiers	$0	$0	$0
Bioreactor	$0	$14,524,084	$14,524,084
Gravity Thickeners	$663,351	$663,351	$663,351
DAF Thickeners	$0	$3,692,278	$3,692,278
Digesters	$8,885,222	$4,442,611	$8,885,222
Mechanical Dewatering	$0	$0	$0
UV	-	-	-
Odour Control System	$500,000	$500,000	$500,000
Site Works	$362,920	$1,614,381	$1,614,381
Admin/Maint. Building	$1,300,000	$1,300,000	$1,300,000
Control System	$421,943	$972,893	$1,150,597
Electrical Substation (allow)	$65,000	$85,000	$75,000
Existing Facility Upgrades	$200,000	$200,000	$200,000
Sub - Total	$16,955,205	$32,551,366	$37,161,682
Division 1 Cost	$322,461	$712,365	$827,623
Engineering	$2,712,833	$5,208,219	$5,945,869
Project Management/QA/QC	$678,208	$1,302,055	$1,486,467
Contingency	$5,086,561	$9,765,410	$11,148,505
Total Capital Cost	$25,756,000	$49,540,000	$56,571,000

TABLE 10.3
LGWWTP ANNUAL OPERATING & MAINTENANCE COST ESTIMATE FOR INTERIM OPTIONS

YEAR	INTERIM		
Option	2031	2031	2031
	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF
O&M COSTS			
Labour	$1,626,268	$1,742,430	$1,742,430
Chemical Costs	$1,604,770	$179,021	$1,653,213
Biolite Replenishment	$0	$27,000	$20,202
Residuals Management	$803,957	$1,562,839	$1,647,160
Energy	$440,880	$564,978	$564,978
Repair/Maintenance	$1,536,117	$1,726,389	$1,782,637
Adminstration and others	$808,525	$745,661	$713,167
Land and Building Lease	$331,839	$331,839	$331,839
Total (O&M Costs)*	$7,153,000	$6,881,000	$8,456,000
Total (O&M Costs)**	$3,005,000	$2,733,000	$4,308,000

Notes

*: Entire plant O/M costs including existing primary plant and upgrade

**: Upgrade O/M costs only (existing primary plant excluded)

TABLE 10.4
LGWWTP LIFE CYCLE COST FOR INTERIM OPTIONS

STAGE	INTERIM		
YEAR	2031	2031	2031
OPTION	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF
Discounted O&M Cost	$13,095,343	$11,910,007	$18,773,623
Discounted Capital Costs	$12,437,680	$23,923,074	$27,318,373
Total Discounted Capital and O & M Costs at present value	$25,534,000	$35,834,000	$46,092,000

There are two paths possible for upgrading through Interim Treatment to Build-out to Secondary. These are: (1) using CEP until 2031 followed by Build-out to Secondary, or (2) constructing 50% ADWF biological treatment as an interim stage. A summary of unit operations requiring upgrading are presented in Table 10.5 showing the development of these paths.

TABLE 10.5
LGWWTP UNIT PROCESS SIZING FOR PREFERRED OPTIONS

	Interim Treatment Using CEP Alone		Interim Treatment Using 50% ADWF BAF		
	Interim	Build-out	Interim		Build-out
	Option 1 CEP	Option3 2xADWF BAF	Option 2A 50% ADWF BAF	Option 2B CEP 50% ADWF BAF	Option 3 2xADWF BAF
Inlet PS Upgrade	-	yes	-	-	yes
Screening Upgrade	-	Yes	-	-	yes
Grit Removal Upgrade 8.5 m dia	0	2	0	0	2
Chemical Dosing	2	2	2	2	2
PSTs	0	0	0	0	0
BAF	0	10	6	6	10
Gravity Thickener 13.7 m dia	1	1	1	1	1
DAF* 15.0 m dia.	0	2	1	1	2
Anaerobic Digesters 22 m dia., 10.1 m depth	2	2	1	2	2
Centrifuge	0	1	0	0	1

Note: Existing plant process units not included above.

*: Redundancy not a concern – could be thickened in PSTs.

10.8 APPROACH TO IMPLEMENTATION

The selection of the preferred option for the interim upgrade has to consider the following factors:

- Final process option selected for the build-out to secondary
- Construction date for build-out to secondary
- Permit compliance for BOD and TSS
- Improvements in LC50 bioassay test results

The methodology for the projections illustrated in Figures 10.1 and 10.2 is discussed in Section 3.1.4 of Appendix 10. Effluent BOD concentration is the critical parameter determining the timing of upgrade, whereas effluent TSS concentration meets the 99% reliability target comfortably.

TABLE 10.6
SUMMARY OF ANALYSIS FOR INTERIM UPGRADES

Interim Option	Effluent BOD & TSS Compliance above 99%	Effluent Toxicity Reduction	Remarks
CEP only	To 2030 for BOD and TSS	33% (based on small scale testing)	• Lowest capital cost • Can be operated intermittently • Allows postponement of biological process • Very high chemical cost
50% BAF with no CEP	To 2030 for BOD and TSS	33% (based on small scale testing	• Double capital cost compared to CEP • O/M cost similar to CEP •
CEP + 50% BAF	To 2030 for BOD and TSS	> 80% (based on small scale testing)	• Generates the largest quantities of sludge • Highest capital and O/M cost • Seasonal CEP may reduce operating costs
All options			• Centrate biological treatment with BAF could reduce BOD and provide test module for full BAF trial

11.0 SOLIDS HANDLING

This chapter summarizes the average annual sludge quantities estimated for interim options for LGWWTP as discussed in Appendix 3 and Appendix 7 (Sections 11.1 to 11.3). These summaries are useful for process comparison. Updated sludge volume estimation is also included in Section 11.4, which is the outcome of preferred options as discussed in Appendix 10. This is developed to assist the GVRD in evaluating options for sludge disposal and reuse.

11.1 ESTIMATED FUTURE SLUDGE QUANTITIES AND QUALITY

The sludge quantity and characteristics are critical in the selection of treatment upgrade processes. Different treatment processes will produce different types of sludge, which require different levels of treatment and handling efforts. Most importantly, the sludge quantity and characteristics will affect the capital investment, O/M cost and beneficial usage opportunities. The following factors are the main considerations:

- Sludge quantity
- Ease of sludge stabilization (e.g. digestion)
- Ease of handling (e.g. dewaterability)
- Nutrient values for land application or other recycling options
- Hygienic quality
- Metals content

The interim upgrade options for LGWWTP identified in Appendix 3 are listed as follows:

- Option 1: Primary + 50% average dry weather flow (ADWF) biological aerated filter (BAF) in Series.
- Option 2A: 50% ADWF RTF + (Q-50% ADWF) Primary (Parallel)
- Option 2B: 100% ADWF RTF + (Q – 100% ADWF) Primary (Parallel)
- Option 3: CEP + 50% ADWF RTF (Parallel)
- Option 4: 50% ADWF HRAS + (Q – 50% ADWF) Primary (Parallel)

11.1.1 Sludge Quantities

The projected sludge production of the interim upgrade options based on average annual load conditions are shown in Figure 11.1 and Figure 11.2 for their dry solids in T/d and wet volume (27 ~ 35% solids concentration) in m^3/year, respectively. Option 3 - CEP + 50% RTF produces the most solids in dry tonnes and the greatest wet sludge volume after dewatering at 35% solids. By 2031, the screening and grit volumes are estimated about 180 tonnes/year and 290 tonnes/year, respectively.

FIGURE 11.1
PROJECTED SLUDGE QUANTITY (DRY SOLIDS) OF LGWWTP

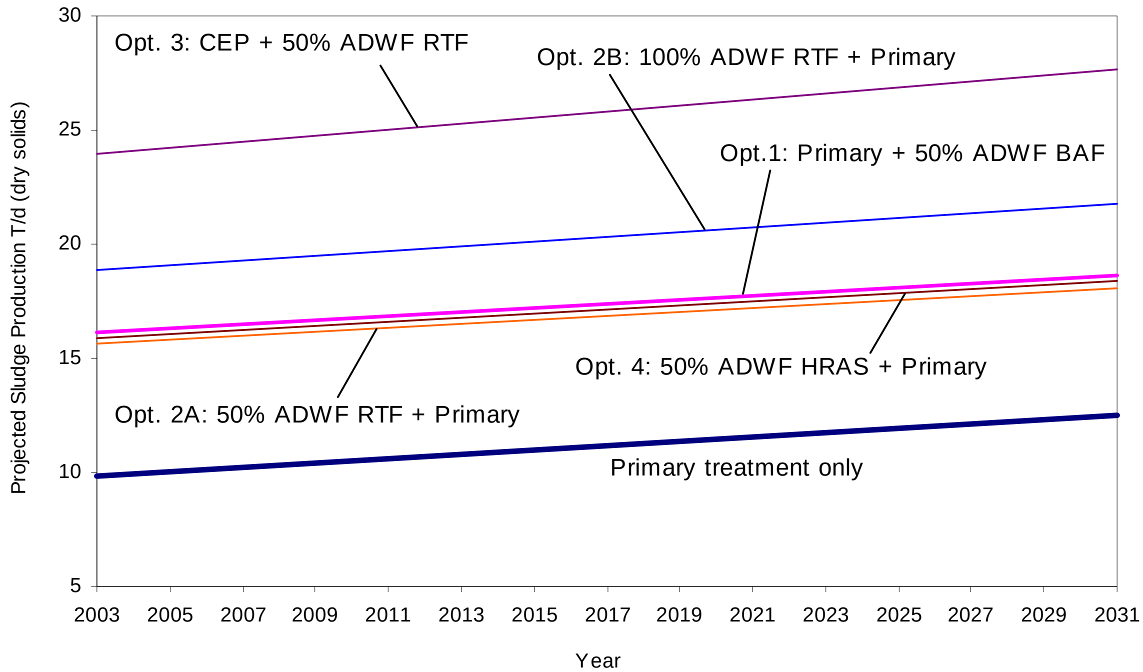
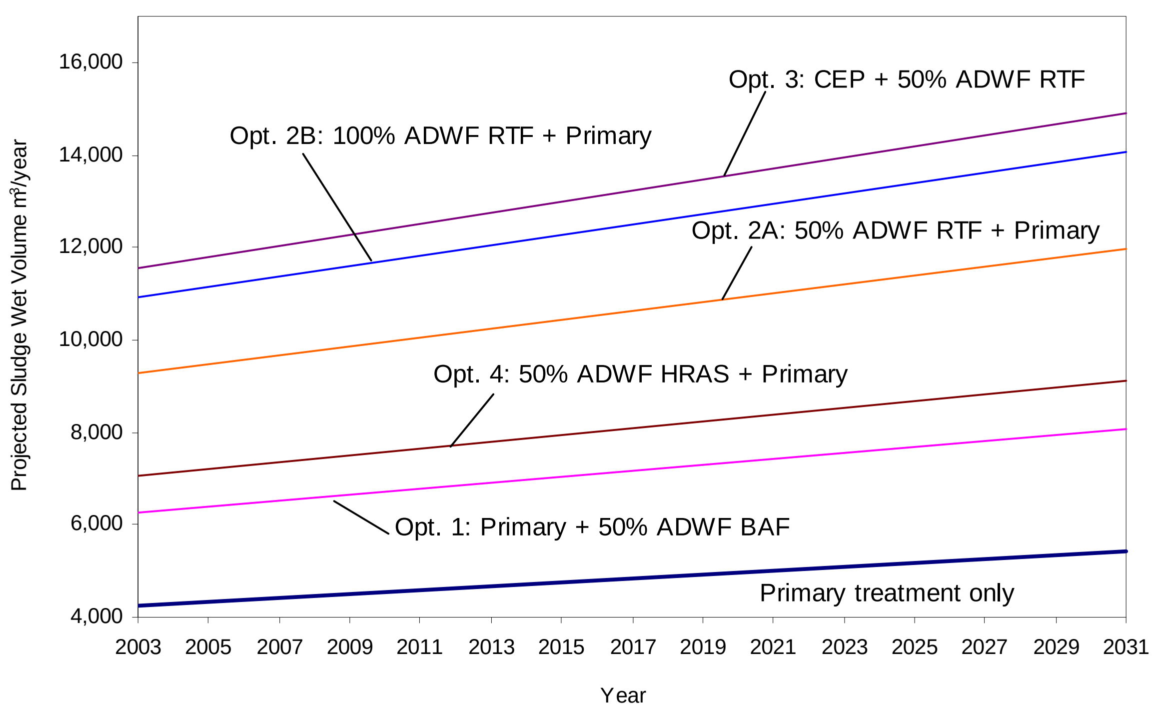


FIGURE 11.2
PROJECTED SLUDGE QUANTITY (WET VOLUME AFTER DEWATERING)



11.1.2 Sludge Quality

The estimated future sludge qualities are summarized in Table 6.1 in Section 6.6.

TABLE 11.1
ESTIMATED FUTURE SLUDGE QUALITY FOR LGWWTP

Chemicals/Nutrients (mg/kg dry kg)	Primary Sludge	CEP Sludge*	Secondary Sludge*
Arsenic Total	1~3	N/A	5~10
Cadmium Total	1~3	10~20	5~10
Chromium Total	30~70	300~400	100~150
Cobalt Total	2~5	N/A	5~10
Copper Total	1,000~1,800	3,000~4,000	2,000~3,000
Lead Total	60~90	400~600	150~200
Mercury Total	5~8	N/A	5~8
Nickel Total	30~50	100~200	50~100
Zinc Total	400~700	1,000~2,000	700~1,500
Total Nitrogen	25,000~40,000	28,000~45,000	30,000~50,000
Total Phosphorus	10,000~20,000	15,000~25,000	20,000~30,000

*: in part based on Bonnybrook WWTP, Calgary, 1998

11.2 SLUDGE STABILIZATION

Current extended thermophilic anaerobic digestion can achieve high degree of VS destruction and pathogen kill. However, the digester capacity needs to be expanded to meet the interim upgrade as indicated in Table 10.5. It is recommended to demolish the #1 and #2 digesters (currently not in use) and release the space for plant expansion. The digester system should be capable of producing different levels of end products, e.g. Class A or Class B for the markets' needs.

Should there be needs to improve sludge stabilization efficiency and reduce/defer digester capacity expansion, the following alternatives can be considered which have been discussed in Section 7 of Appendix 6:

- Temperature phased anaerobic digestion (TPAD)
- Acid-gas phased anaerobic digestion (AGAD)
- Extended thermophilic anaerobic digestion

11.3 SUMMARY OF RECOMMENDATION FOR INTERIM SLUDGE MANAGEMENT

Mechanical dewatering and hauling to offsite locations are considered the most economic option for sludge handling during the interim stage at LGWWTP. Due to the space constraint, there is no space onsite for storage or stockpiling. The recommended interim sludge handling strategies are:

- Maximize the process capacity of existing treatment units, including the gravity thickener, anaerobic digester at thermophilic operating condition, and centrifuge dewatering (e.g. extending operation hours).
- Add extra sludge handling capacities, including additional gravity thickener (for redundancy capacity), DAF (for waste activated sludge thickening), thermophilic anaerobic digesters, and centrifuge.
- Design/retrofit the digester system to be capable of being operated to produce different quality requirements for biosolids recycle options (e.g. composting, pelletization, energy recovery etc.) and land applications (e.g. silviculture and mining site reclamation etc.).
- Investigate the economics and feasibility to operate the digester system in a more efficient mode, e.g. staged operation such as temperature-phased digestion.

11.4 CANDIDATE PROCESS IMPLICATIONS ON ANNUAL SLUDGE VOLUMES

The estimated annual sludge volume produced for preferred options discussed in Appendix 10 are summarized in Table 11.2.

This is a refinement of the annual sludge volume indicated in Figures 11.1, 11.2 in Appendix 7.

The average annual sludge volume of the preferred option is estimated based on the design case and the following parameters which reflect the actual plant conditions of LGWWTP:

- Sludge VSS/TSS Ratio: 90% (Primary sludge)
70% (Secondary sludge)
- VS Destruction in Thermophilic Anaerobic Digesters: 65%
- Solids contents of Sludge Cake: 35% (Primary sludge)
27% (Combined sludge)

TABLE 11.2
LGWWTP ANNUAL SLUDGE PRODUCTION FOR PREFERRED OPTIONS

YEAR		INTERIM			BUILD-OUT
Option	Unit	2031	2031	2031	2046
		CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Raw Sludge/Biosolids					
Primary Sludge	T/d	16	12	-	13
CEP Sludge	T/d	3	0	19	0
Secondary Biosolids	T/d	0	12	7	12
Total Raw Sludge	T/d	19	24	25	26
Thickened Sludge					
Gravity Thickener (5%)	m^3/d	372	239	372	269
DAF (3.5%)	m^3/d	0	333	198	350
Total Thickened Sludge	m^3/d	372	572	569	619
Digested Sludge	m^3/d	372	572	569	619
Dewatered Sludge	m^3/d	22	43	45	57

It should be noted that Options 2A (50% BAF without CEP) and 2B(CEP + 50% BAF) produce a similar amount of sludge. This is because CEP sludge has a slightly larger VSS to TSS ratio.

PART 3 – NORTH SHORE SEWAGE TREATMENT ALTERNATIVES

12.0 SUMMARY OF RELOCATION OPTIONS FOR NORTH SHORE

12.1 OPTION 1 – LIONS GATE EXPANSION ON EXISTING SITE

12.1.1 Land Use and Site Location

The 3.4 ha site is leased from the Province of British Columbia and from the Vancouver Port Authority. It is bounded by rail tracks, by Burrard Inlet and by Squamish First Nations lands.

12.1.2 Site Access

Access is currently over rail tracks and through First Nations lands.

12.1.3 Water Table

Much of the existing plant is only 2 m above the extreme Burrard Inlet HHW level. Almost all tanks are subjected to hydrostatic uplift pressures, a factor which will have to be considered in the design and construction of new plant.

12.1.4 Geotechnical Issues

The site is prone to long-term settlement and liquefaction during earthquake events. The former condition can be addressed by preloading and piling design. To address lateral soil movement and liquefaction, it is proposed to densify the ground along the water and to install soil anchors on the existing plant.

12.1.5 Odour

The plant is adjacent to a shopping mall and a residential area. Odours are a concern but this can be minimized by appropriate treatment.

12.1.6 Visual Treatment

The location is exposed. This would be taken into account during the design of the expansion through architectural design for the new building enclosures.

12.1.7 Site Expansion

By the selection of appropriate technology, it would be possible to provide treatment capacity beyond the year 2046 on the existing site. This option has the attraction that the site is currently utilized for wastewater treatment.

12.2 OPTION 2 – NEW SITE FOR LIONS GATE WWTP

It would be feasible to construct a new treatment plant to the east of the existing Lions Gate WWTP, which would replace the existing plant. For further details refer to Section 8.5.

12.3 OPTION 3 – MULTIPLE PLANTS

Treatment of sewage generated on the North Shore could be carried out at three dispersed sites, the existing LGWWTP and at two other plants. For further details refer to Section 8.6.

13.0 OPTION 4 - DIVERSION TO NORTH SHORE TO IONA ISLAND

13.1 OPTIONS FOR CROSSING BURRARD INLET

Initially four options for the crossing of Burrard Inlet were considered.

- Route 1
  Across Burrard Inlet west around Point Grey, tying in to the headworks of the Iona Island Wastewater Treatment Plant (IIWWTP).

- Route 2
  Across Burrard Inlet/English Bay tying into the Highbury Interceptor at 1st Avenue.

- Route 3
  Under Stanley Park connecting to the Jervis Sewage Pump Station and from there through an upgraded pumping system to the 8th Avenue Interceptor.

- Route 4
  Across the waters of Vancouver Harbour to the Columbia Street Pump Station and from there through upgraded pumps to the 8th Avenue Interceptor.

Routes 3 and 4 had previously been considered by others and discounted. These options were revisited and at a concept cost estimate level were confirmed as being much more costly than Routes 1 or 2. Routes 3 and 4 are discounted from further consideration.

Route 2 was subsequently subdivided into two sub-options.

- Route 2a)
  Across Burrard Inlet/English Bay tying into the Highbury Interceptor at 1st Avenue.

- Route 2b)
  Similar to 2a) except parallel this route with a new tunnel or pump station forcemain combination to the IIWWTP.

13.2 GEOTECHNICAL CONSIDERATIONS FOR A MARINE CROSSING

Further geotechnical investigation is required, including a detailed assessment of the potential for liquefaction and submarine mud slides.

The presence of boulders and cobbles will make construction difficult and may preclude the use of trenchless methods.

Across the Inner Harbour and Outer Harbour the depth of marine clay can exceed 62 metres.

Slope stability will need to be assessed and will have an impact on the route selection.

Detailed design should take into account the effects of long term and differential settlement along the pipeline alignment.

13.3 OPTIONS FOR LAND PORTION

Various North Shore Diversion Options were evaluated as follows:

- Pumping from LGWWTP to north end of Highbury Interceptor.
- 3 metre diameter tunnel parallel to Highbury Interceptor.
- 6 metre diameter tunnel parallel to Highbury Interceptor.
- shallow bury forcemain above the Highbury Interceptor and to convey 2 x ADWF from the North Shore.
- Shallow bury forcemain above the Highbury Interceptor to convey 2 x ADWF from the North Shore plus the ADWF from the 8th Avenue Interceptor.

Present worth costs, including a consideration for O&M and using a 6% discount factor, were used in comparing these land options.

All options commence their discharge from the Lions Gate Wastewater Treatment Plant and the flow would be pumped to the north end of the Highbury tunnel area. Present worth costs of these options are summarized in Table 13.1.

TABLE 13.1
COST ESTIMATES OF NORTH SHORE DIVERSION OPTION (INCLUDING O&M COSTS)

Option	Description	NPV in Millions of Dollars(Notes)
1	Pumping from LGWWTP to north end of Highbury Interceptor	\$30.1
2	3 m tunnel parallel to Highbury Interceptor	\$55.9
3	6 m tunnel parallel to Highbury Interceptor	\$64.4
4	Shallow bury forcemain above Highbury system for 2 x ADWF	\$44.4
5	Shallow bury forcemain above Highbury system for 2 x ADWF plus ADWF from 8th Avenue Interceptor.	\$64.4

Notes:

1) All costs include pumping from the Lions Gate plant to the north end of the Highbury tunnel area.
2) Costs include the following allowances:
   - Division 1 Cost 2.5%
   - Engineering 16%
   - Project Management/Quality Control 4%
   - Contingency 30%

The options provide alternate schemes, some of which would convey greater flows than others. In selecting a preferred option, consideration will have to be given to the magnitude of the cost of each option and of other factors as part of the triple bottom line assessment that combines economic, environmental and social factors.

13.4 EXPANSION OF IONA ISLAND PLANT FOR NORTH SHORE FLOW

The diversion of the flow from the North Shore to Iona Island based on diverting two times the ADWF would require the following work at Iona Island based on using the TF/SC process.

TABLE 13.2
IMPACT OF NORTH SHORE DIVERSION
ON IIWWTP BUILD-OUT TO SECONDARY

Unit Process/Flow	VSA Flows Only	Combined NSSA and VSA Flows
Flow in Primary Plant (MLD)	1,530 MLD	1,738 MLD
Flow in Secondary Plant in 2036 (MLD)	912 MLD	1,120 MLD
Headworks	No expansion proposed	Additional
Primary clarifier	No additional units	3 additional unit
Trickling Filter	6 units	7 units
Solids Contact Tanks	4 units	5 units
Final Clarifiers	16 units	20 units
Gravity thickeners	No additional units	1 additional unit
DAF Thickeners	3 units	4 units
Digesters	4 units	5 units or 4 larger units

The additional cost to provide the additional capacity for the North Shore flow is estimated at $96.6 million for both the primary and secondary treatment as well as solids handling. In addition, the headworks at Iona Island would have to be expanded to deal with the increase in flow unless, grit removal and screening is provided at the Lions Gate site prior to pumping (screening and degritting as pre-treatment for the North Shore flow).

13.5 PRELIMINARY COST ESTIMATE OF DIVERSION OPTIONS – SENSITIVITY ANALYSIS

13.5.1 Conveyance from North Shore to Iona Island

An analysis of the net present value costs indicates that the capital component is dominant. Even for the option assuming the pumping of 2 x ADWF from the North Shore plus the ADWF from the 8th Avenue Interceptor the energy cost contributes less than 25% to the total NPV at a 6% discount rate.

The capital cost estimate for the conveyances have a relatively high level of uncertainty because of the difficulty of assessing accurately the cost of laying pipelines in a trench excavated across the Burrard Inlet and of driving a tunnel from the West Point Grey area to the Iona Island treatment plant.

Even greater uncertainty is associated with the options assumed for analysis. These offer varying levels of service and facility and impact the existing systems to different extents.

13.5.2 Treatment at Iona Island WWTP

The cost of increasing capacity to treat the flows from the North Shore can be estimated with greater accuracy than the cost of conveyance.

13.5.3 Overall Assessment

The concept of conveying flows from the North Shore to Iona Island WWTP has greater technical uncertainty than the alternative of upgrading the existing plant. Energy cost is only a significant concern if the most energy intensive options are contemplated.

14.0 TRIPLE BOTTOM LINE AND SENSITIVITY ANALYSIS

14.1 FACTORS CONSIDERED

An analysis was carried out to evaluate the options for sewage treatment for the NSSA including various locations for the Lions Gate plant and the diversion of Lions Gate flows to Iona Island plant for treatment. The options for sewage treatment for the North Shore are described in Section 8. The factors considered include cost, sustainability/environmental, and social issues:

1. Cost Factors

   - Capital cost – Total capital cost including construction, engineering and contingency.
   - Operating and maintenance cost – Operating cost of existing primary plant plus additional cost for the upgrade.
   - Lifecycle cost – Lifecycle cost include both capital cost and operating and maintenance cost and is a measure of all future cost expressed in today's dollars, i.e. net present value (NPV).

2. Sustainability/Environmental

   - Compliance with water quality objectives – Risk to water quality in case of plant failure or rupture of underwater pipeline.
   - Risk to marine organisms – Risk in case of plant failure or rupture of underwater pipeline.
   - Risk of odours if odour control system fails – Risk refers to the comparable generation of odours of the processes and their dependence on odour control.
   - Production of aerosols – Production of small droplets which can be carried off site by wind.
   - Visual impact – Size and profile of above ground tankage and structure
   - Energy use – Includes the energy used for pumping.
   - Impact on wildlife habitat – Impact on wildlife habitat by new plants and construction activities of diversion pipelines.
   - Seismic concerns – Regarding the possible failure of facilities during a major earthquake.

3. Social

   - Navigation issues – Applicable to the construction of new outfalls and diversion pipelines.
   - Approvals – Refers to approvals for new sites for sewage treatment, new outfalls and marine pipelines.
   - Traffic generation – Refers to truck traffic during construction and for hauling of biosolids.
   - Land acquisition – Existing land use and process required to change land use to a sewage treatment facility.
   - Public acceptance – Refers to public acceptance of new North Shore sites for one or several sewage treatment plants.

- Leasing issues – Ownership of existing Lions Gate site.
- Construction issues – Impact of construction activities on residences and businesses.

14.2 ASSESSMENT OF OPTIONS

A summary of the capital and life cycle costs and a triple bottom line assessment of the four major options for treating flows from the NSSA are detailed in Table 14.1.

The four options considered are:

1. Expansion of the Lions Gate plant on the existing site.
2. Multiple North Shore WWTPs.
3. Relocate to a new site on the North Shore
4. Combing NSSA and VSA flows at Iona Island. Five sub-options were identified for conveyance of flow from the North Shore to Iona Island.

The rating under a Triple Bottom Line (TBL) assessment was derived by assigning a score of 3 to Option 1 as the baseline (including cost, environmental, and social categories). Other options are rated against this baseline with a "4" indicating a more adverse situation, and a "5" indicating much more adverse situation. A "2" indicates a better situation and a "1" indicates a very much better situation. The option obtaining the lowest total score is considered the most favorable option in the triple bottom line analysis.

TABLE 14.1
SUMMARY OF COSTS AND TRIPLE BOTTOM LINE ASSESSMENT

	Summary of Costs			**Triple Bottom Line Assessment Scores**			
Option		**Capital**	**LC NPV**	**Cost**	**Env.**	**Social**	**Total**
1	**Expansion of Lions Gate**	**$107 M**	**$35 M**	**3**	**3**	**3**	**9**
2	**Multiple North Shore WWTP's**	**$185 M**		**5**	**4**	**4**	**13**
3	**Relocate North Shore WWTP**	**$160 M**		**5**	**3**	**4**	**12**
4	Combining VSA and NSSA at Iona Island						
	Iona Island treatment capacity expansion (A)	$97 M	$59 M				
	Conveyance Cost (B)						
	• Option 1	$124 M	$30 M				
	• Option 2	$238 M	$56 M				
	• Option 3	$276 M	$64 M				
	• Option 4	$170 M	$44 M				
	• Option 5	$221 M	$64 M				
	Combined cost of conveyance & Iona Island WWTP treatment capacity expansion (A+B)						
4.1	**• Option 1**	**$221 M**	**$89 M**	**5**	**4**	**3**	**12**
4.2	**• Option 2**	**$335 M**	**$115 M**	**5**	**4**	**3**	**12**
4.3	**• Option 3**	**$373 M**	**$123 M**	**5**	**4**	**3**	**12**
4.4	**• Option 4**	**$267 M**	**$103 M**	**5**	**4**	**3**	**12**
4.5	**• Option 5**	**$318 M**	**$123 M**	**5**	**4**	**3**	**12**

The assessment as presented in Table 14.1 is discussed under each category below:

14.3 DISCUSSION

14.3.1 Cost

The table summarizes the capital and NPV costs as derived for each option in the body of the report. It can be seen that the cost of upgrading the existing plant on its present site (Option 1) is considerable lower than that of any other option. For this reason, this option is used as the base case against which all other options are compared to complete the triple bottom line analysis.

14.3.2 Environmental

Option 2 - Multiple North Shore WWTPs

The discharge of flow in the embayed region of Burrard Inlet, where the flushing action is less than at the present discharge point, may increase the impact of the effluent on the local environment. Treatment to remove phosphate could be required. Because phosphorus removal is a more advanced treatment process, there is a greater risk of non-compliance in the effluent quality. The current location appears to have minimal impacts on the environment but the environmental risks at the other locations are not known. For these reasons, the environmental risk in Option 2 is assessed to be higher than in Option 1.

Option 3 – Relocate North Shore WWTP

The relocation to a site close to the existing treatment plant has no impact on the environment other than during construction phase. The ratings of two sites (existing and a new site) are considered to be equal.

Option 4 – Combine NSSA and VSA at Iona Island

The principal environmental risk associated with all the sub-options (4.1 ~4.5) is that of conveying sewage across the Burrard Inlet. The impact is associated with the construction of the pipelines, which have to be laid in a trench excavated in the floor of the inlet. The risk associated with pumping and conveying large flows over long distances is for accidental spills to the ocean should the system fail. Failure could result from pump station outages or from damage to the pipeline by earthquake, ships anchoring, and corrosion over time. Treatment at Iona Island is not considered to offer any significant environmental advantage. The rating awarded reflects the higher risk for all crossing options.

14.3.3 Social

Option 2 - Multiple North Shore WWTPs

The location of the required treatment capacity on a number of sites is considered to have a negative impact on a greater number of people because of the odours and the traffic generated by the facilities. This is reflected in the rating given.

Option 3 – Relocate North Shore WWTP

The relocation of the treatment plant would impact persons who are established around the new site and on the traffic routes to the site. People being aware of the existence of the present plant have had the opportunity to choose to locate elsewhere.

Option 4 – Combine NSSA and VSA at Iona Island

Despite treating 2 x ADWF at Iona Island, it would be necessary to construct a wet weather treatment plant on the North Shore to treat flows greater than 2 x ADWF. Some land could be made available at the existing site for alternative use. The impact of this plant would be similar to the present plant.

14.4 OVERALL ASSESSMENT

Based on the assumptions set out above, the ratings indicate that Option 1 (expanding the plant on the existing site) is considered better than, or at least equal to, all the other options in all (cost, environment, and social) categories.

It is recognized that the assessment is highly subjective and that by changing the emphasis on the various factors influencing the rating of the options, a different conclusion could be reached. The conclusion reached must therefore be considered tentative and may need to be re-examined.

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 1
Domestic and Non-Domestic Trucked Liquid Waste

FINAL REPORT

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1 INTRODUCTION

The objectives of this study are to assess the Truck Liquid Waste (TLW) facility condition at the Iona Island Wastewater Treatment Plant (IIWWTP) and Annacis Island Wastewater Treatment Plant (AIWWTP), including the following:

- Evaluate the existing facilities and operational conditions
- Determine the non-domestic TLW pre-treatment efficiency
- Assess the TLW impacts on the main treatment process at the IIWWTP and their influences on effluent quality
- Future planning strategy for regional TLW treatment (including the option of relocating the non-domestic TLW facilities to the AIWWTP)

A TLW sampling program was developed in this study to confirm the TLW characteristics and efficiency of pre-treatment at the IIWWTP. The sampling results were used in a mass-balance analysis to assess the potential impacts of TLW on the plant effluent quality. A survey was conducted at the IIWWTP to collect feedback from the TLW haulers/drivers for future improvement considerations. The TLW sampling, survey summary, and mass balance analyses are included in the appendices.

1.1 BACKGROUND

The trucked Liquid Waste (TLW) is categorized into two groups by its generation:

(1) Domestic TLW, which is collected from septic tanks, holding tanks, portable toilets, and other domestic sources, and

(2) Non-domestic TLW, which is generated in the industrial practices such as poultry and food processing, or the sources other than the domestic TLW such as restaurants grease traps and commercial businesses.

The TLW generated in the GVRD is predominately treated at two regional wastewater treatment plants, including the Iona Island (IIWWTP) and the Annacis Island (AIWWTP). The Northwest Langley (NWLWWTP) accepts only the holding tank TLW collected within the Township of Langley, which represents a small portion of flow and loads.

The TLW facilities at IIWWTP are equipped to accept both domestic and non-domestic TLW in two separate receiving systems. The domestic TLW is discharged directly into the influent pipe without any pre-treatment. The non-domestic TLW is pre-treated by screening and gravity sedimentation, before entering the primary sedimentation tanks (PST) of the main sewage treatment system.

A new septic receiving station with coarse screening was reopened at the AIWWTP in November 2002, and only the domestic TLW is allowed at this new facility. The TLW (holding tank waste) received at the NWLWWTP is discharged directly into the sludge digester without pre-treatment.

According to the GVRD By-Law 164, registration is required for industries and hauling services to ship their TLW to GVRD facilities for treatment. Registration includes a record of the TLW generation, volume and typical waste characteristics. Upon their hauling to the treatment facilities, the TLW volume and type of waste generated are registered and the disposal fees are charged based on the volume only, regardless of the type of waste.

The solids and organic concentrations in the TLW can be hundreds or thousands folds higher than in the domestic sewage. Due to the high waste strength of the TLW, it was postulated that the discharge of TLW without proper pre-treatment could cause significant impacts on the main treatment process, especially to systems with primary treatment only, such as the IIWWTP. The compliance priorities for the period of interim treatment include effluent toxicity reduction and BOD/TSS removals. The presence of oxygen depleting substances, or the BOD/soluble BOD in the effluent has been identified as the major cause of toxicity failures in the 100% effluent 96-hour LC_{50} tests (EVS, 2001). Also the treatment efficiency of the TLW pre-treatment facility has not been evaluated. The TLW may have significant impacts on the treatment efficiency and effluent quality.

1.2 SCOPE OF WORK

The scope of work in this study includes:

- Review the existing facilities relative to their location, operation and effectiveness
- Using existing reports and data quantify and characterize existing and future liquid waste streams. Also conduct a sampling program to confirm the TLW quality and quantity characteristics and to determine the effectiveness of the TLW facility
- Determine the TLW pre-treatment efficiency
- Assess the TLW potential impacts on the main treatment process
- Propose alternative TLW handling options if necessary

2 EXISTING TLW PRE-TREATMENT FACILITY

The existing TLW pre-treatment facilities at IIWWTP and AIWWTP, as well as flow and load conditions, are discussed in Section 2.1 and Section 2.2 respectively. The regional TLW generation loads are projected in Section 2.3.

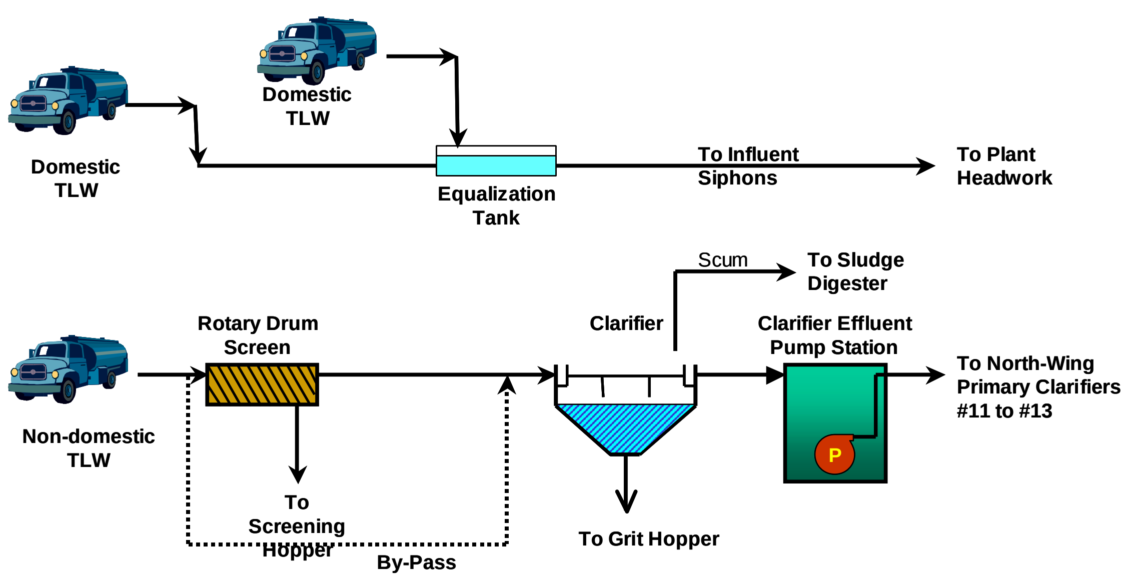
2.1 IONA ISLAND

2.1.1 Operation Hours and Process Description

The TLW operation hours at IIWWTP are from 5:00 am to 6:00 pm on weekdays and Saturdays. The TLW haulers are required to report to the registration office while arriving the plant, and fill their manifests to detail the source of liquid waste and volume. The trucks are then directed by the plant staff to the domestic or non-domestic discharge stations based on the TLW sources of generation.

Currently, the domestic and non-domestic TLW are handled differently as shown on process schematics illustrated in Figure 2.1. The domestic TLW is discharged directly into the plant influent siphon pipes by gravity flow, through a manhole and a holding tank. The holding tank was operated to pace the TLW flow into the main treatment process. However, the holding tank is not currently in use because of frequent clogging problems associated with the existing 4” pipeline. The domestic TLW received at IIWWTP is introduced into the headwork and treated in conjunction with the main treatment process. Three grit dump stations (two in operation one standby) are provided for the trucks to hose down grit and residuals in the hauling tanks for the protection of the bar screens and influent pumps in the headwork.

FIGURE 2.1
PROCESS SCHEMATIC OF TLW PRE-TREATMENT AT IIWWTP



The non-domestic TLW is received and pre-treated by a stand-alone system located at the northwest corner of the plant site located between the digester No. 4 and pipe gallery. The pre-treatment system consists of a cylindrical rotary drum screen (9 mm perforated openings) and an 8-m diameter circular clarifier. A default 6" flexible quick connector is normally used by the haulers, however different sizes of connectors and hoses are also provided on-site, The TLW is first pumped through the rotary drum screen to remove large screenings and debris. The screening rejects are washed and pressed by hydraulic rams to reduce the volume, then collected in a disposal bin. The screenings are handled together with the plant bar screening residuals for final disposal.

After the rotary drum screen, an 8 m diameter (3.5 m of side-wall depth) clarifier is operated to remove floatable scum, fat, oil and grease (FOG), and settleable solids. The floatable material is collected from the surface of the clarifier with a mechanical scraper, and then pumped to the digesters for further treatment. To prevent the FOG accumulation, a stream of hot digester supernatant is introduced to the floatable scum hopper to flush down and dissolve the FOG.

The settleable solids are collected at the bottom of the clarifier, then pumped to the plant grit hopper for further dewatering. As with the grit produced at the plant, the settled solids are disposed of at landfill. The bottom settleable solids are withdrawn on a weekly basis. Currently, the bottom solids withdraw is carried out during the weekends and the solids level in the clarifier is approximately 30% to 35% of the total clarifier depth.

The treated TLW effluent is directed through a 1.2 m underflow baffle to the submersible pump station, and then pumped to the north influent channel entering the plant pre-aeration tanks. The discharge location of the TLW effluent is located between Tank #10 and #11 in the north influent channel, therefore the TLW effluent is likely distributed to Tank #11, #12 and #13 only.

This TLW pre-treatment facility is also operated to treat the primary sedimentation tank scum flow when there is no TLW being processed. A manual control is provided for the operators to switch from the normal scum pump mode to the TLW receiving mode. Currently, the pre-treatment is operated to receive the primary scum flow about 75% of the time.

2.1.2 Flow and Load

A monthly summary of TLW hauling frequency and volumes are listed in Table 2.1 (October 2002 to May 2003, GVRD Facility Summary, 2003). The total volumes received during this period at IIWWTP were about 53,172 m^3 of domestic TLW and 6,268 m^3 of non-domestic TLW, respectively. All of the non-domestic TLW and approximately 80% of the domestic TLW generated in the region were delivered to IIWWTP during November 2002 to May 2003. Compared with the records of the previous year (October 2001 to May 2002), the domestic and non-domestic TLW volumes received at IIWWTP decreased by 12% and 38%, respectively. Considering the domestic TLW received by the new facility at AIWWTP, the total domestic TLW volume collected in the region actually increased by 4%, compared to the same period of previous year.

In the 2002 and 2003 records, more than 70% of the truckloads arrived at the IIWWTP between 7:00 am to 1:00 pm. The daily average truckloads arriving at the plant were about 24 trucks of domestic TLW and 4 trucks of non-domestic TLW.

TABLE 2.1
IIWWTP TLW VOLUME MONTHLY RECORDS

Source	Domestic TLW			Non-domestic TLW		
Month	Trucks	Total Volume, m^3	m^3/truck	Trucks	Total Volume, m^3	m^3/truck
Oct-02	958	9,315	9.7	129	832	6.4
Nov-02	791	6,774	8.6	117	757	6.5
Dec-02	626	4,921	7.9	110	850	7.7
Jan-03	709	6,423	9.1	156	970	6.2
Feb-03	657	5,853	8.9	112	763	6.8
Mar-03	724	6,972	9.6	103	712	6.9
Apr-03	419	6,262	14.9	116	660	5.7
May-03	737	6,652	9.0	120	724	6.0
Average	703	6,647	9.7	120	783	6.5

The most recent sampling data for the TLW are documented in a 1997 report (GVRD 1997), which identifies the two different source categories of TLW and their respective weighted averages:

- Domestic TLW (septic tank, holding tank, portable toilet and others)
  BOD: 1,560 mg/L
  SBOD: 550 mg/L (approximately 35% of BOD)
  TSS: 5,060 mg/L
- Non-domestic TLW (beverage, fish, fruit/vegetable, poultry, meat, waste reduction, restaurant and others)
  BOD: 41,000 mg/L
  SBOD: 14,500 mg/L (approximately 35% of BOD)
  TSS: 101,000 mg/L

The Trucked Liquid Waste Facility Review Draft (GVRD 2002) reported 1,980 mg/L BOD and 2,190 mg/L TSS of a pre-treatment effluent composite sample. Discrete samples of BOD at the same day ranged from 3,630 mg/L to 11,800 mg/L, however no time-series concentration profiles were provided. The sampling program developed in this study is to obtain the raw TLW and the pre-treatment effluent concentrations. The sampling results are detailed in Section 3.0.

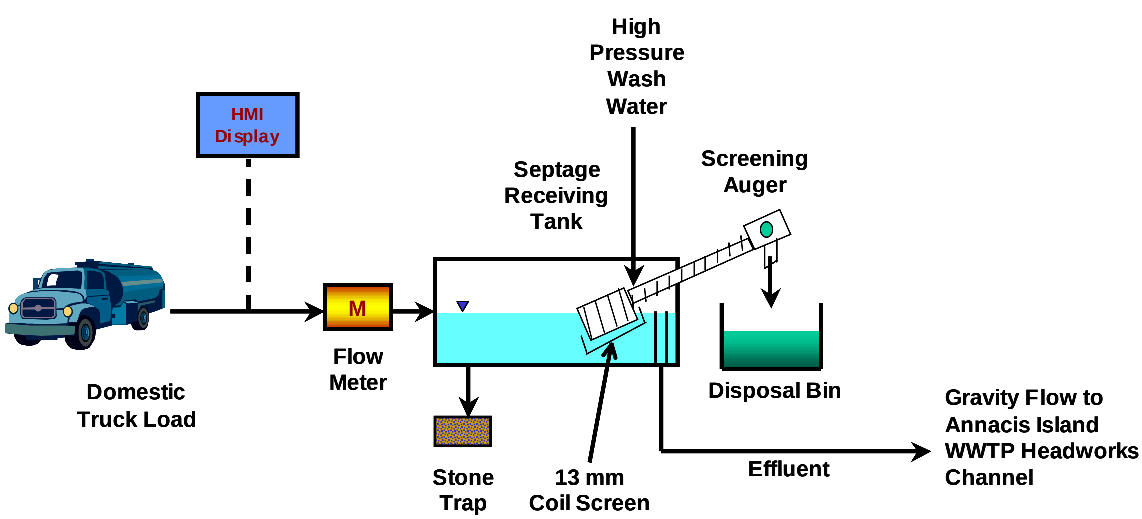
2.2 ANNACIS ISLAND

2.2.1 Operation Hours and Process Description

A septic receiving station with a pre-treatment facility was commissioned in late November 2002 to receive domestic TLW at AIWWTP. The operating hours of the TLW receiving station were from 5:00 am to 3:00 pm during weekdays and half day on Saturdays. The hours of operations have been extended since early August of 2003, and the facility is currently available for discharge from 5:00 am to 5:00 pm Monday to Saturday, but still restricted to the domestic TLW only.

The receiving station and the pre-treatment process schematic are illustrated in Figure 2.2. Instead of manned attendance as at the IIWWTP, the AIWWTP has an automatic registration system including card lock access, video surveillance, and touch-pad human-machine interface (HMI) registration. Two 6" flexible quick connectors are provided at the station and the operation is prompted by the HMI instructions. To initiate the discharge operation, the haulers are requested to report the TLW generator (by municipality) and source of TLW (holding tank, septic tank, portable toilet, or stormwater from residents). The TLW volume is recorded automatically with in-pipe magnetic flowmeters. The on-line registration information is recorded and managed at the GVRD head office.

FIGURE 2.2
PROCESS SCHEMATIC OF TLW PRE-TREATMENT AT AIWWTP



The TLW is first pumped into the septage-receiving tank equipped with a Rotamat® type stationary semi-cylindrical coil bar screen basket (13-mm openings) and rotary cleaning racks. The rejected screenings are transported to the dewatering classifier by a shaftless spiral, high-pressure water wash, and collected in a disposal bin. Plastic wrap is used to retain the compressed screenings to prevent spill and odour. A grit/stone trap is equipped at the bottom of the receiving tank to collect coarse settleable materials. The treated TLW is directed to the plant headwork channel by gravity flow.

Plant staff reported satisfactory operation of the existing TLW receiving and pre-treatment facility, except the under-sized grit/stone trap required frequent cleaning (several times a week). More use of this receiving facility can be expected due to the operation hour extension from 3:00 pm to 5:00 pm since August 2003.

2.2.2 Flow and Load

A monthly summary of TLW hauling frequency and volumes are listed in Table 2.2 (November 2002 to May 2003, GVRD Facility Summary, 2003). The peak truckloads arrived at the plant between 6:00 am to 7:00 am in the morning and 12:00 noon to 2:00 pm. The AIWWTP TLW facility was designed to handle 50% of the regional domestic TLW, but only about 20% of the domestic TLW generated in the region were hauled to AIWWTP during period from November 2002 to May 2003. The District has not enforced any restriction on the disposal locations (either to IIWWTP or AIWWTP) and allows the haulers to choose their routes based on their operational convenience. The extension of hours may encourage more TLW to be delivered to AIWWTP.

TABLE 2.2
AIWWTP TLW MONTHLY RECORDS

Source	Domestic TLW		
Month	Trucks	Total Volume, m^3	m^3/truck
Nov-02	57	396	7.0
Dec-02	134	1,389	10.4
Jan-03	160	1,593	10.0
Feb-03	126	1,308	10.4
Mar-03	161	1,691	10.5
Apr-03	170	1,677	9.9
May-03	195	1,911	9.8
Average	143	1,424	9.7

No TLW characteristics and pre-treatment efficiency assessments have been carried out at the AIWWTP facility. Since only the domestic TLW is allowed at AIWWTP facility, the TLW characteristics can be assumed to be similar to the domestic TLW at IIWWTP. In comparison with the main treatment capacity at AIWWTP (secondary treatment with trickling filter and solids contact), 50% of the regional domestic TLW represents less than 1% of TSS and 0.5% of BOD loads. The TLW flow is considered negligible compared to the plant wet weather hydraulic capacity (1,090 ML/D). No significant TLW flow and load impacts are expected at the AIWWTP.

2.3 REGIONAL TLW PROJECTIONS

The TLW TSS and BOD loads generated in the region are estimated using the methodology developed in Appendix 3. The TLW flows are considered negligible in the projection since they present less than 0.05% of the total annual flow in Vancouver Sewer Area (VSA). The estimated per capita BOD and TSS unit rates of the regional contributing population for all domestic and non-domestic sources are compared as follows:

- Domestic TLW
  TSS: 0.003 kg/cap·d
  BOS: 0.001 kg/cap·d
- Non-domestic TLW
  TSS: 0.006 kg/cap·d
  BOD: 0.003 kg/cap·d

The projections of the TSS and BOD loads are illustrated in Figure 2.3 and 2.4 respectively, showing the upper envelope (1.0% population growth rate) and lower envelope (0.6% population growth rate). The population growth rate projections after 2021 decrease to 0.21% and 0.09 % for the upper and lower envelopes respectively. In 2021, the BOD and TSS loads of TLW are projected to increase by 10 ~ 20% from the current level. No consistent trends were concluded from the historical records of 1997 to 2001 (GVRD, 2002). The actual TLW generations are subject to the industrial operations such as seasonal variations, business development, as well as municipal by-law enforcements such as pre-treatment requirements, source control, and sewer connections. Special event such as Olympic Game in 2010 may also generate more TLW in the region for a short period. These factors should be considered in the operational arrangement and future planning.

FIGURE 2.3
TLW TSS LOAD PROJECTION IN GVRD

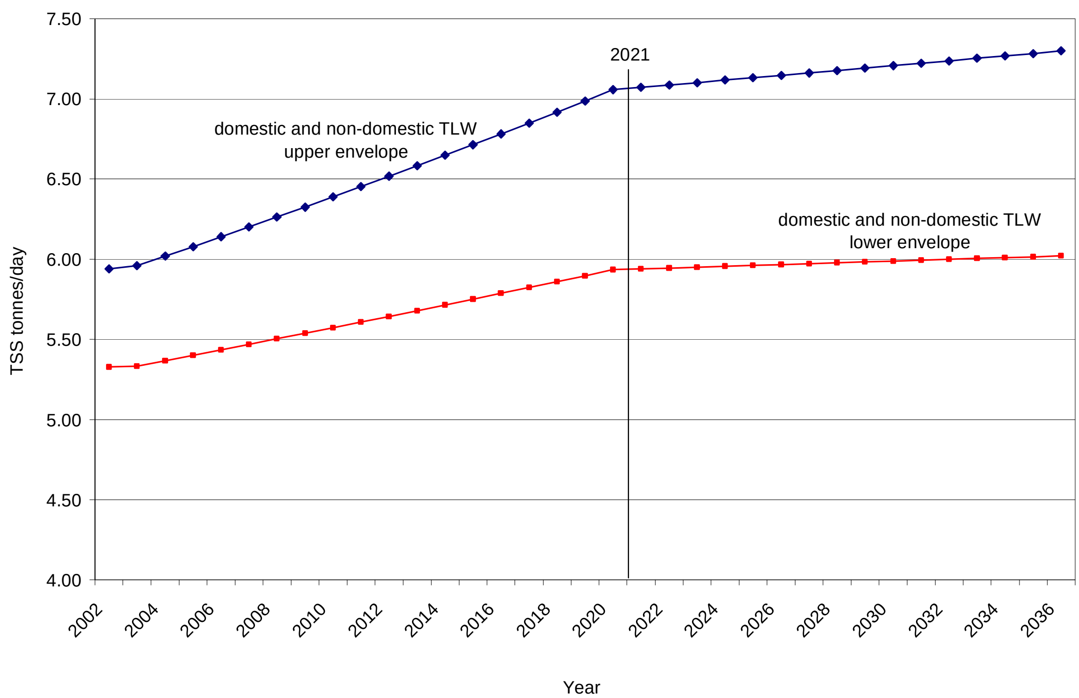
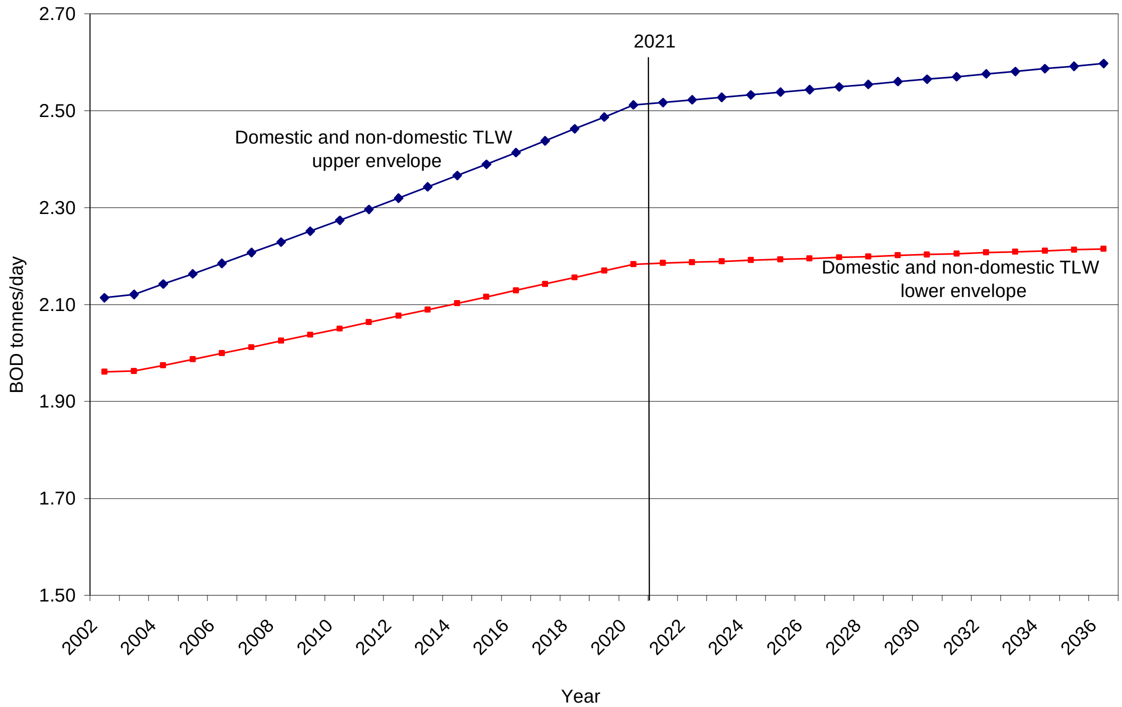


FIGURE 2.4
TLW BOD LOAD PROJECTION IN GVRD



3 TLW SAMPLING AND SURVEY

A TLW sampling program was developed to gather information on the TLW characteristics and pre-treatment efficiency. The sampling program and procedures are detailed in Appendix A. If necessary, a future sampling program can be considered through the enforcement of GVRD By-Law 164. This By-Law authorizes the Manager to request intermittent sampling and analysis of the waste discharges at the discharger's expense. A questionnaire was also prepared to survey the TLW haulers/carriers for their feedback about the current TLW service and possible relocation of the TLW facility in the future. A blank questionnaire is included in Appendix B. The survey was conducted during August 2003 and the questionnaires were completed when the TLW haulers reported to the attendant office for registration.

3.1 TLW SAMPLING AND RESULTS

3.1.1 TLW Characteristics

The sampling results of the raw TLW characteristics (the volume-weighted averages of BOD, SBOD and TSS of S1 samples) are summarized in Table 3.1, together with a comparison of the 1997 sampling results (GVRD, 1997). The volume-weighted average BOD concentration of the domestic TLW (septic tank, holding tank, and portable toilet) found in this study (August 2003) are similar to the 1997 results. However, the TSS concentration in this study was about double. Compared with the septage characteristics reported in the USEPA report of BOD (440 ~ 78,600 mg/L) and TSS (1,100 ~ 130,500 mg/L), the concentrations found in this study were within the typical ranges (USEPA, 1994).

The SBOD/BOD ratios found in this August 2003 sampling were between 0.16 ~ 0.45 with an average of 0.30. The 1997 GVRD sampling study also reported an SBOD/BOD ratio averaged about 0.35 (GVRD 1997). Both numbers in 1997 and August 2003 samplings were higher than the SBOD/BOD ratio about 0.10 reported in the USEPA study (USEPA 1994).

Some waste categories of non-domestic TLW were not available during this sampling period. Some waste categories were no longer being delivered to IIWWTP for discharge, such as beverage and fruit/vegetable TLW, because there are other disposal options available in the Fraser Valley area. Some waste sources are not consistently hauled to the plant due to the seasonal operation of the industries, e.g. fish and poultry TLW. The restaurant TLW was found to be the most dominate non-domestic TLW during this sampling period.

Due to different TLW sources and varied industry operation modes, TLW concentrations varied significantly. In comparison with the 1997 results, the non-domestic TLW waste strength also increased substantially. The weighted averages of TSS and BOD found in this August 2003 sampling program were almost double the concentrations obtained in 1997.

TABLE 3.1
SAMPLING RESULTS OF TLW CHARACTERISTICS
(VOLUME-WEIGHTED AVERAGES)

Year	1997			2003			
Parameters	BOD	SBOD	TSS	BOD	SBOD	TSS	Number of Samples
Units	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	
Domestic TLW							
Septic tank	5,784	-	32,963	3,870	760	65,790	3
Holding tank	409	-	784	630	170	3,580	6
Portable toilets	2,971	-	18,354	5,140	1,730	21,820	5
Weighted average	1,560	550*	5,060	1,570	460	10,520	-
Non-Domestic TLW							
Beverage	55,600	-	14,500	-	-	-	-
Fish	28,133	-	27,406	7,400	3,600	18,100	1
Fruit and Vegetable	68,786	-	273,263	-	-	-	-
Poultry	38,537	-	83,896	335,000	12,000	463,000	2
Meat	20,388	-	114,900	17,570	2,190	51,910	2
Waste reduction	72,457	-	73,833	194,230	36,860	407,160	6
Restaurant	32,521	-	146,800	118,320	64,400	140,690	6
Weighted average	41,000	14,500*	101,000	121,750	21,460	210,640	-

*: approximately 35% of BOD

3.1.2 TLW Pre-Treatment Effluent Quality and Efficiency

The pre-treatment effluent quality (S2 samples) at IIWWTP was monitored on August 8th, August 19th, and August 29th, respectively, with discrete samples taken every hour from 7:00 am to 4:00 pm (a two-hour interval was used on August 8th). The BOD, SBOD and TSS concentrations are illustrated in Figure 3.1, Figure 3.2, and Figure 3.3, respectively. These results indicated that the pre-treatment effluent qualities were significantly affected by the TLW discharges. The peak concentrations were found to be correlated directly to the TLW discharge events.

FIGURE 3.1
NON-DOMESTIC TLW PRE-TREATMENT EFFLUENT BOD CONCENTRATIONS

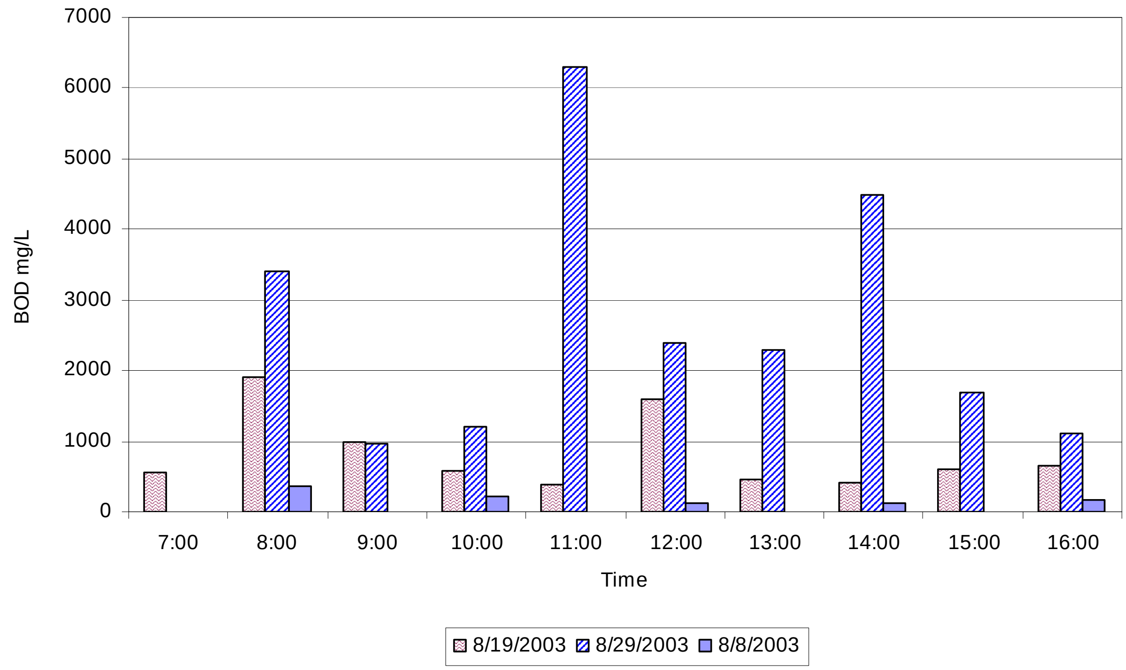


FIGURE 3.2
NON-DOMESTIC TLW PRE-TREATMENT EFFLUENT SBOD CONCENTRATIONS

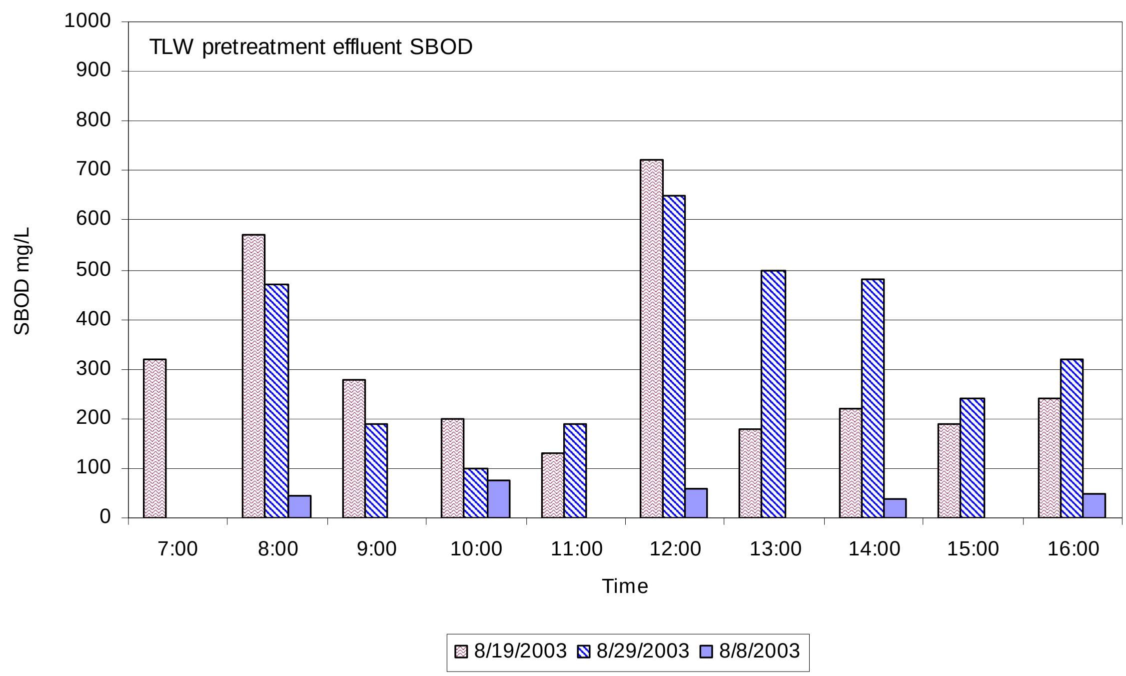
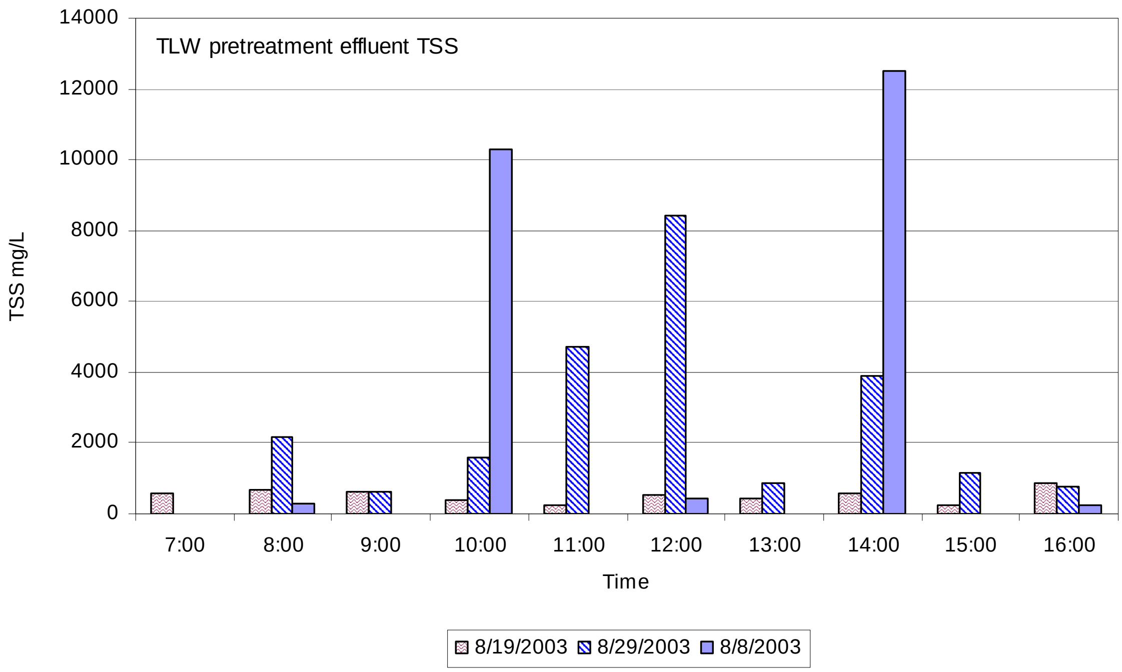


FIGURE 3.3
NON-DOMESTIC TLW PRE-TREATMENT EFFLUENT TSS CONCENTRATIONS



Since the withdrawals of the settleable solids from the TLW pre-treatment clarifier were arranged on a weekly basis, the wet volume of the settleable solids was estimated at about 60 m^3 per week. The settleable solids samples (S3) TSS concentrations averaged about 164,000 mg/L. The scum pump flow rate and non-domestic TLW discharge rates were estimated between 1.2 m^3/min to 2.8 m^3/min, which results in the hydraulic retention (HRT) in the TLW clarifier varying from approximately 2.4 to 1.0 hr.

Due to the nature of the TLW discharges and operational arrangements, the estimation of the TLW clarifier removal efficiency is difficult to approximate. In this sampling study, the BOD removal efficiencies in the TLW clarifiers were estimated about 20 to 80%, and negligible for SBOR removal. The TSS removal efficiencies ranged between 30 to 60%. An average TSS removal efficiency about 45% was estimated based on the weekly mass balance calculations. The pre-treatment efficiency was highly dependent on the waste characteristics, such as the settleable solids fraction, settleable BOD fraction, and organic degradation rates, and TLW discharge volumes. The dilution factor due to the primary scum flow and the clarifier volume were also significant.

The results obtained from this August 2003 sampling program were used to provide the TLW concentration profiles needed for the mass-balance evaluation detailed in Section 4.0.

3.2 TLW SERVICE SURVEY AND RESULTS

The TLW service survey was carried out at IIWWTP during August 7th to August 27th, with plant staff assistance. The questionnaires included provision for comments about the TLW service. The survey was completed by the TLW haulers at the registration office. About 70% of the haulers attending the IIWWTP TLW facility handed in their answers, which resulted in 33 valid survey sheets. The survey results and comments are summarized as follows:

1. Is the current TLW operation hours suitable to your hauling operation?

   - Iona Island: YES (79%) NO (21%)
   - Annacis Island: YES (50%) NO (50%)

   More than three quarters of the drivers responded that the current operation hours at IIWWTP are suitable to their operation, and half of the drivers felt that the hours at AIWWTP are suitable. It should be noted that the operation hours at AIWWTP was extended a few days before the survey was carried out and some of the drivers may not have been aware of this change. In general, the haulers requested longer operation hours at both plants for their convenience. It may be possible at AIWWTP since a keyless entrance and HMI registration systems are in place. However there is no immediate assistance available at AIWWTP if the haulers experience difficulty with the facility. This would require multiple shifts of staff at IIWWTP to provide extended operation hours since there is only one shift of staff currently.

2. Have you experienced waiting for discharge for more than 10 minutes at a single trip?

   - Iona Island: Never (12%) Occasional (85%) Very often (3%)
   - Annacis Island: Never (55%) Occasional (35%) Very often (10%)

   Fewer haulers experienced queuing at AIWWTP than at IIWWTP. This was probably due to that only about 20% of the domestic TLW were delivered to AIWWTP. The queuing at IIWWTP mainly occurred at the non-domestic discharge site, however, it seemed not to be a serious problem since there were averaged about 4 truckloads every day.

3. What type of waste do you haul/carry at this trip?

   - Domestic waste: (88%)
   - Non-domestic waste: (21%)
   - Both domestic and non domestic: (9%)

4. If only the Annacis Island is open for the DOMESTIC disposal, will it affect your operation?

   - NO (27%)
   - YES (73%)

5. If only the Annacis Island is open for the NON-DOMESTIC disposal, will it affect your operation?

   - NO (48%)
   - YES (52%)

   73% and 52% of the participants answered that their operations will be affected if the AIWWTP is the only facility available for domestic and non-domestic TLW discharge, respectively. Three major concerns were:

   a) traffic condition (21 out of 33),
   b) operation hours (19 out of 33), and
   c) distance to AIWWTP (15 out of 33).

Comments provided by the survey participants are summarized as follows:

- Longer operation hours at both plants were requested (12 out of 33 responses), e.g. 24 hours a day and 7 days a week, or extended daytime hours from 5:00 am to 6:00 pm.

  Currently, the operation hours at IIWWTP and AIWWTP are close to the haulers' expectations of daytime hours (5:00 am to 6:00pm) except the weekends. A 24h-7d schedule could provide additional convenience for the haulers. However, this is not practical as it would increase the operational costs particularly at IIWWTP. Plant safety and emergency responses would also be required if extended operation hours are considered after the normal daytime hours. Experimental run of 24h-7d schedule can be planned at AIWWTP to extend the operation hours since the automatic registration and self-service operation is in place. This arrangement may also encourage the haulers to use the facility at AIWWTP.

- Provide a washout facility at AIWWTP for tank cleaning purpose, since IIWWTP already has three grit dumping stations for washout (6 out of 33 responses).

  Installation of a washout facility at AIWWTP can be considered for the convenience of the TLW haulers' operation. The washout facility, e.g. a hose down ramp and grit dump, can be used to dispose off any residuals in the hauling tanks. It may also prevent TLW cross-contamination and encourage the haulers to use the AIWWTP facility. The treatment facilities (pre-screen and headwork) can be protected from tearing and wearing if rocks and some settleable materials can be removed in advance.

- If IIWWTP was no longer available for domestic and non-domestic TLW disposal, more than 50% of the drivers responded that their operations would be affected due to travel distance and traffic condition to AIWWTP, as well as the operation hours.

  These results suggested the location and traffic condition (i.e. Highway 91 and Alex Fraser Bridge interchanges) to the Annacis Island might have played a significant role on haulers' decision to come to AIWWTP or IIWWTP TLW facilities. These factors should be considered in the future planning to maximize the use of AIWWTP facility or a new facility sited elsewhere.

In general, the users were satisfied to have two disposal sites (Iona Island and Annacis Island) for their choices. Longer facility operational hours were preferred at both sites, which may provide flexible hours to their hauling schedule. Queuing for discharge at two disposal sites does not seem to be a serious problem at current truckload conditions. A lack of wash down facility at AIWWTP may have caused inconvenience to the haulers' operation. Hauling distance, traffic conditions and plant operation hours were the key concerns to the haulers.

Extended service hours at both disposal sites would result in additional capital and operational costs, which need to be further justified, such as TLW load increases and TLW impact to the plant. The cost increase will eventually be passed on to the users (i.e. TLW fee schedule). The plant may benefit by having the loads being spread out over a longer period of time.

4 TLW IMPACT MASS BALANCE

Of the TLW delivered to IIWWTP, the annual flow was estimated to be less than 0.05% and the annual loads were estimated less than 4% of total plant loads, respectively (GVRD 2002 and GVRD 2003a). However, the instantaneous impacts resulting from the TLW discharge volume and frequency have not been monitored and determined. For operational management and planning purposes, a mass balance analysis was carried out to assess the potential impacts of the TLW on the primary influent loads, operational condition, and effluent quality at IIWWTP.

For assessing the instantaneous impacts of the TLW at IIWWTP (including the domestic and non-domestic TLW), the following conditions and operational arrangements were considered:

- The TLW receiving facilities are open for disposal only during weekdays 6:00 am to 5:00 pm.
- The characteristics of TLW varied greatly because of different generation sources and industry operation patterns. The weighted average concentrations of TLW characteristics were used in the preliminary assessment (GVRD 1997 and GVRD 2003a), and the preliminary results were presented in Workshop No.1 at GVRD (June 24, 2003) to justify the sampling program development in this study. The TLW characteristics determined in this study (Section 3.1) were further applied in a modified mass-balance, which is detailed in this section.
- The TLW concentration profiles for BOD, SBOD and TSS were established based on the sampling results, representing the typical TLW discharge scenarios for the domestic and non-domestic TLW.
- The domestic TLW is pretreated along with the plant sewer influent by bar screens and aerated grit chamber. The domestic TLW characteristics are represented in the influent samples taken at the influent pump chamber. The non-domestic TLW is however, treated separately by the pre-treatment facility with a rotary drum screen and gravity clarifier.
- The pre-treatment effluent of non-domestic TLW (supernatant from the clarifier) is discharged into the North Influent channel at the point between the primary clarifier # 10 and #11. The TLW effluent was assumed to be equally distributed into tanks #11, #12 and #13. These three tanks were expected to receive higher loads than the other primary clarifiers (#1 ~ #10, and #14~#15) during the TLW operation hours.
- Uneven flow distribution in the IIWWTP primary settling tanks has been identified in previous studies (CH2M Gore & Storrie 1996 and Hay & Company 2002). The situation is reported to be worst during the low flow condition, mainly due to the hydraulic configuration and the numbers of influent pumps in operation.
- The performance of the primary sedimentation tanks is dependent on the influent hydraulic and solids loads, particle size and weight distribution; therefore, the primary treatment efficiency could be affected by the TLW contributions and overall flow conditions.

- The hydraulic characteristics of the narrow primary effluent channels (north and south channels) may result in poor particle suspension, and the effluent sample concentrations (flow-rate proportional composite) may overestimate the true removal effectiveness of the primary settling tanks.

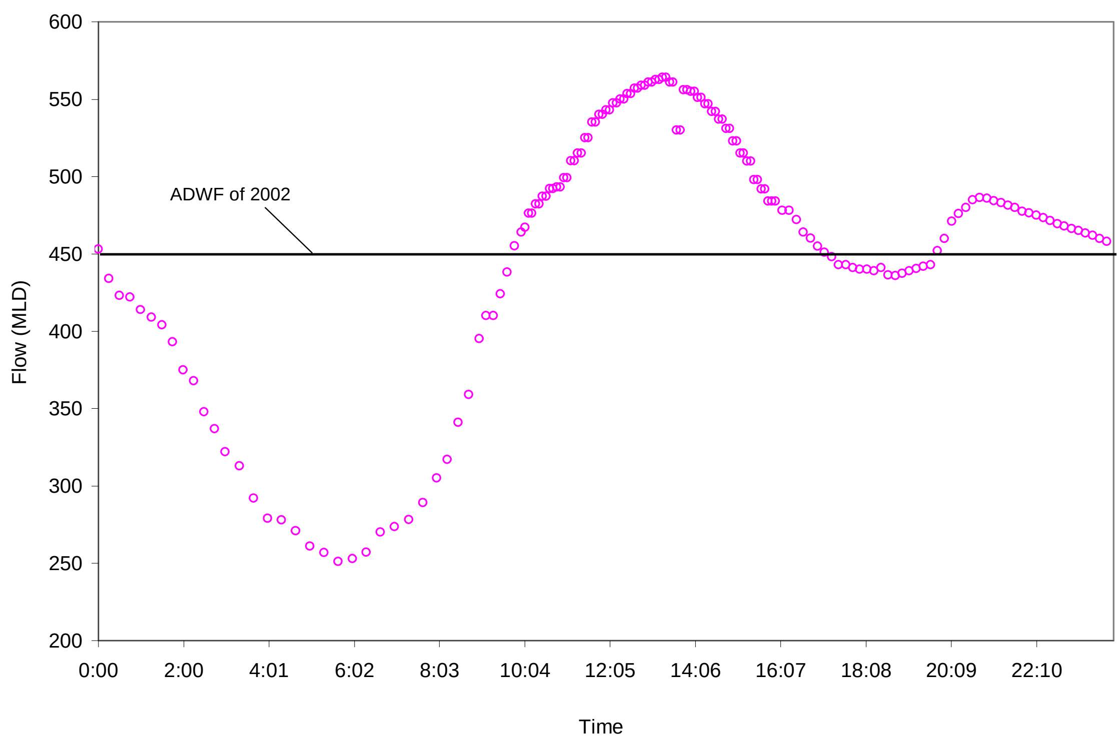
Due to these unique conditions and operational arrangements, some assumptions are used for the flow conditions, TLW concentration profiles, and treatment efficiency in the mass balance exercise.

4.1 FLOW DATA AND ASSUMPTIONS

4.1.1 Plant Influent Diurnal Flow and Concentrations

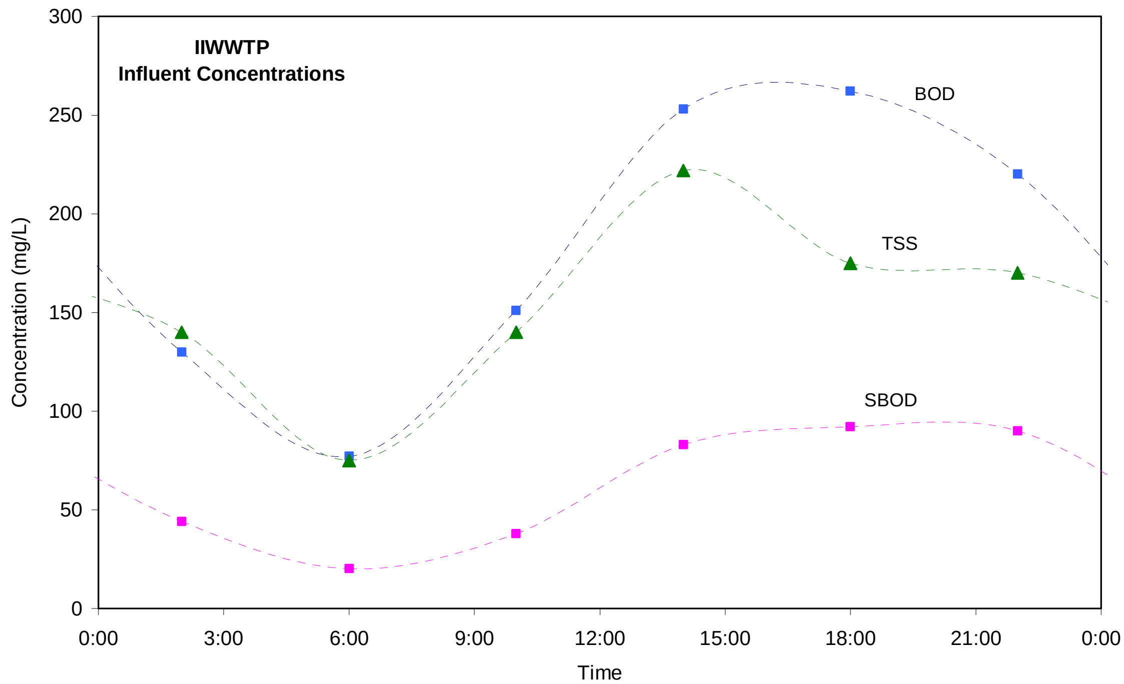
The diurnal flow and concentration profiles of dry weather condition were adopted from previous studies (CH2M Gore & Storrie 1996 and GVRD 2003a). The diurnal flow ratios and ADWF of 2002 (450 ML/D) were applied to develop the instantaneous flow rates of a typical dry weather day shown in Figure 4.1. Two peak flow rates were identified at about 1:30 pm and 20:30 pm. The highest and lowest flow rates were approximately 1.33 and 0.58 times of the average flow, respectively. The symbols shown on Figure 4.1 also indicate a typical sampling schedule of the flow-rate proportional composite sampling setup.

FIGURE 4.1
IIWWTP INSTANTANEOUS INFLUENT FLOW RATES OF A TYPICAL DRY WEATHER FLOW DAY



The BOD, soluble BOD and TSS concentration profiles during a typical dry weather flow day are shown in Figure 4.2, assuming smooth transition between each sampling points (CH2M Gore & Storrie 1996). Similar daily diurnal variances were observed in both the concentration and flow rate profiles. The highest concentrations were found in the afternoon around 2:00 pm to 3:00 pm, which were also around the time of highest flow rate entering the plant.

FIGURE 4.2
IIWWTP INFLUENT CONCENTRATIONS OF BOD, SBOD AND TSS



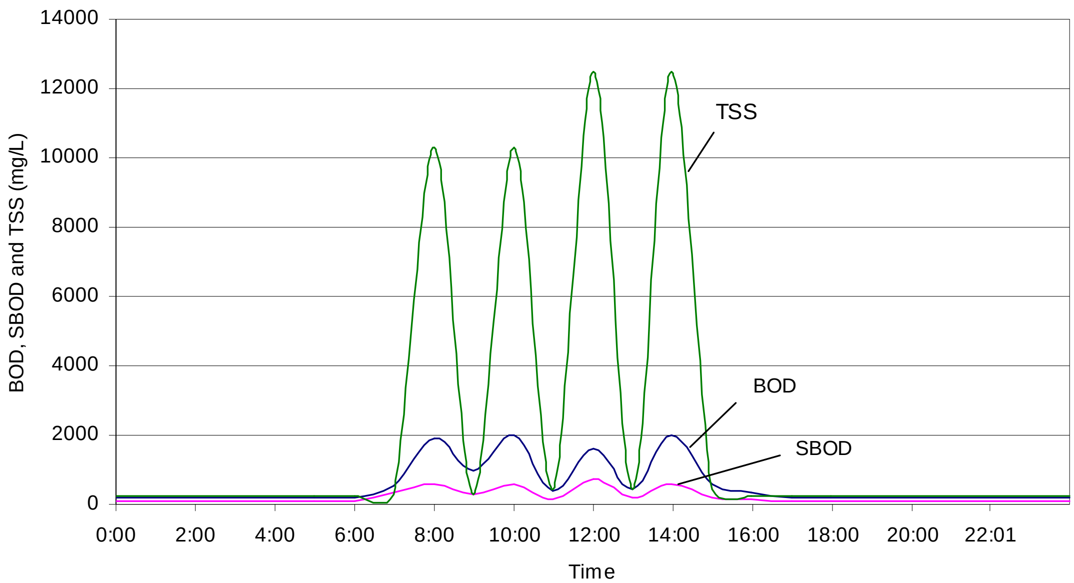
According to a GVRD investigation (GVRD 2003a), the internal recycling and side streams have been included in the influent flow and constituent concentrations, including the domestic TLW, airport influent, screening return, lagoon return, thickener supernatant, and plant drains. These recycling and side streams ranged approximately 0.05 to 1.4% of the total plant influent flow, and 0.8 to 1.3% of the total plant BOD load. Based on the annual average conditions, the domestic TLW was found to be less than 0.05%, 0.7% and 2.7% of total flow, BOD and TSS loads, respectively, (GVRD 2002). The non-domestic TLW is not included in the plant influent flow and constituent concentrations.

4.1.2 IIWWTP TLW Flow and Concentrations

In this study, the maximum monthly total discharge volumes (October 2002) of the domestic and non-domestic TLW were adopted. The maximum daily deliveries were 31 trucks per day. The non-domestic truckloads averaged 4.5 trucks per day and the domestic truckloads averaged 26.5 trucks per day. The domestic TLW discharge rates were estimated about 0.5 to 3.0 m^3/min.

In the mass balance exercise, the weighted averages of domestic TLW derived from the sampling results (Table 3.1) were used in the calculations. The concentration profiles of BOD, SBOD and TSS (TLW pre-treatment clarifier effluent) were established in Figure 4.3, simulating the four (4) truckloads of non-domestic discharge during the operation hours. The TLW loads are the sum of the products of respective discharge volumes and constituent concentrations.

FIGURE 4.3
TLW PRE-TREATMENT EFFLUENT QUALITY



4.1.3 <u>Plant Sampling Schedule and Frequency</u>

As required by the plant permit criteria for the IIWWTP, the plant adopts a flow-rate proportional composite sampling schedule and frequency at the influent and effluent pump chambers. The sampling frequency is based on the flow rate proportions and the composite samples are analyzed for reporting purpose. The sample volume of each sampling event is set at 100 mL to make up a daily composite sample total volume less than 20 liters. A conceptual sampling schedule is illustrated in Figure 4.1, with the following sampling frequency:

- 12 AM – 3 AM: every 15 min
- 3 AM – 8 AM: every 20 min
- 8 AM – 9 AM: every 15 min
- 9 AM – 10 AM: every 10 min
- 10 AM – 4 PM: every 5 min
- 4 PM – 12 AM: every 10 min

4.1.4 Primary Influent Flow Distribution and Concentrations

The primary influent flow was assumed to be equally distributed into fifteen primary sedimentation tanks. According to the records, even flow distribution will mostly occur during the high flow condition in a day (Hay & Company, 2002), and the TLW may incur more significant impact during the high flow condition. The non-domestic TLW was assumed to be equally distributed into #11, #12 and # 13 primary sedimentation tanks.

The primary influent concentrations of BOD, SBOD and TSS, with or without the TLW contributions (domestic and non-domestic) are shown in Figure 4.4, 4.5, and 4.6, respectively. The curves with the non-domestic TLW contribution represent the concentrations entering the primary sedimentation tanks #11 to #13. The curves without non-domestic TLW represent the concentrations entering the rest of tanks (#1 to # 10, and #14 to #15). The curves without domestic and non-domestic TLW reveal the primary influent concentrations without both domestic and non-domestic TLW discharges.

FIGURE 4.4
PRIMARY INFLUENT BOD CONCENTRATIONS WITH AND WITHOUT TLW

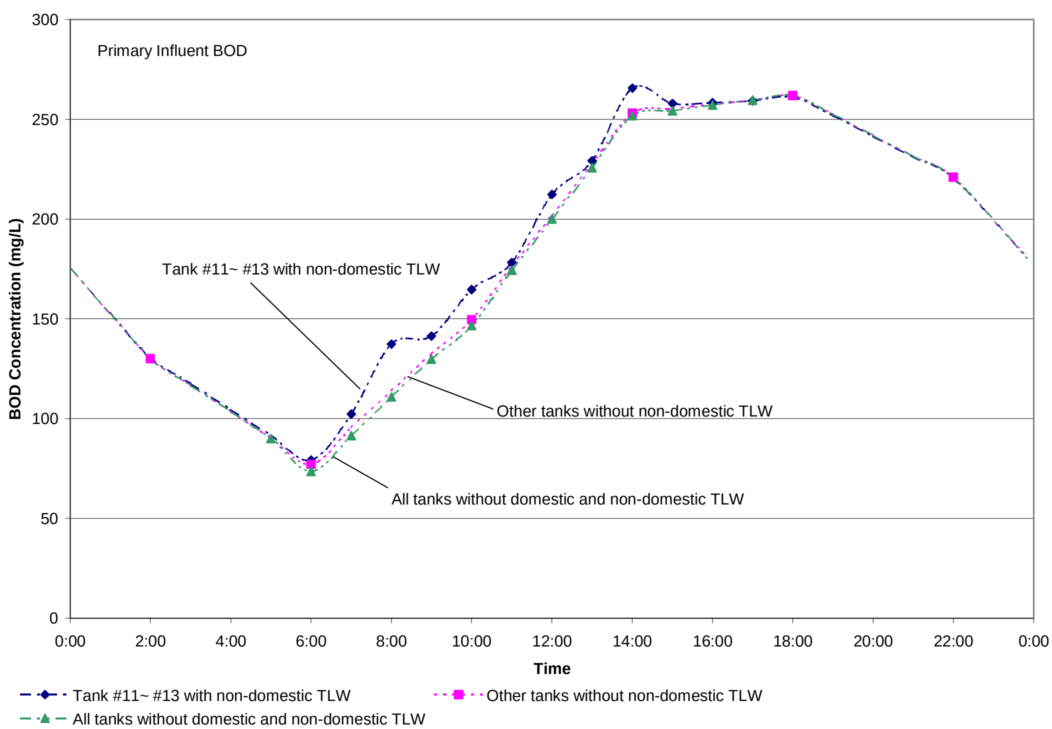


FIGURE 4.5
PRIMARY INFLUENT SBOD CONCENTRATIONS WITH AND WITHOUT TLW

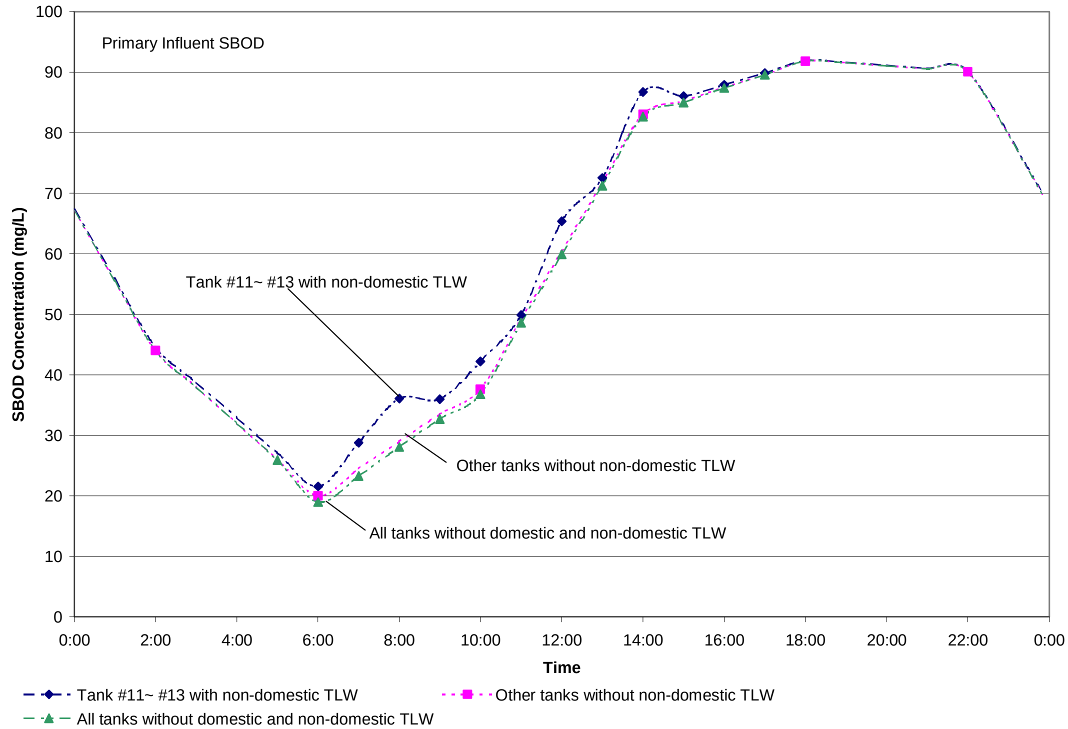
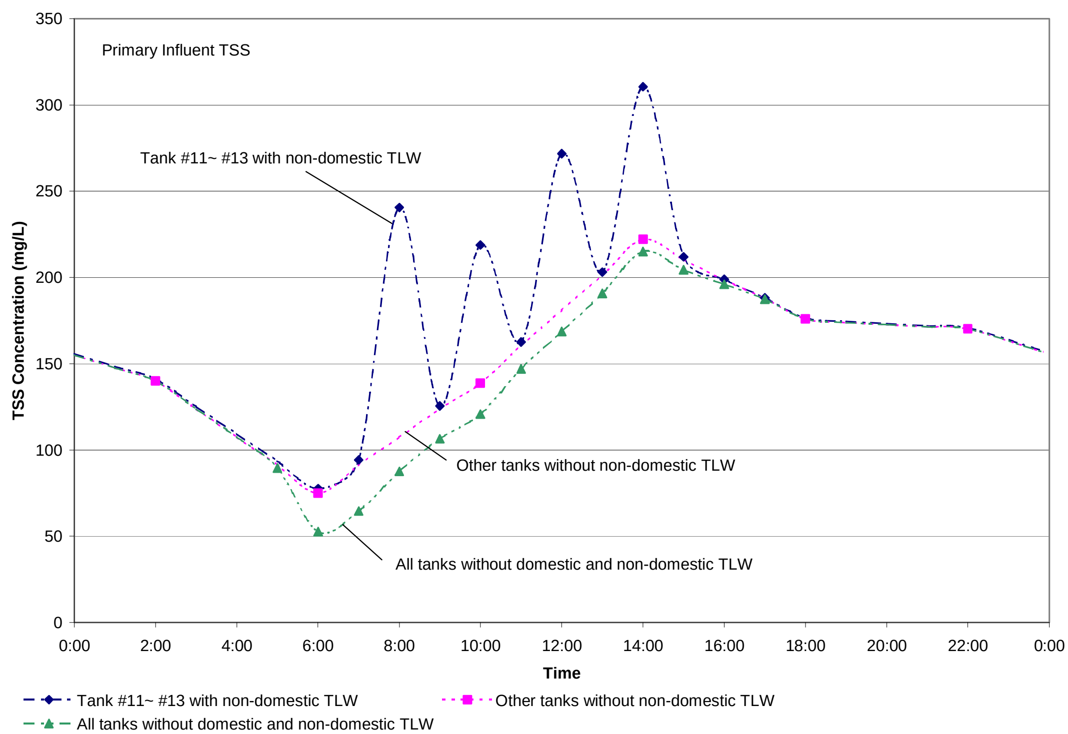


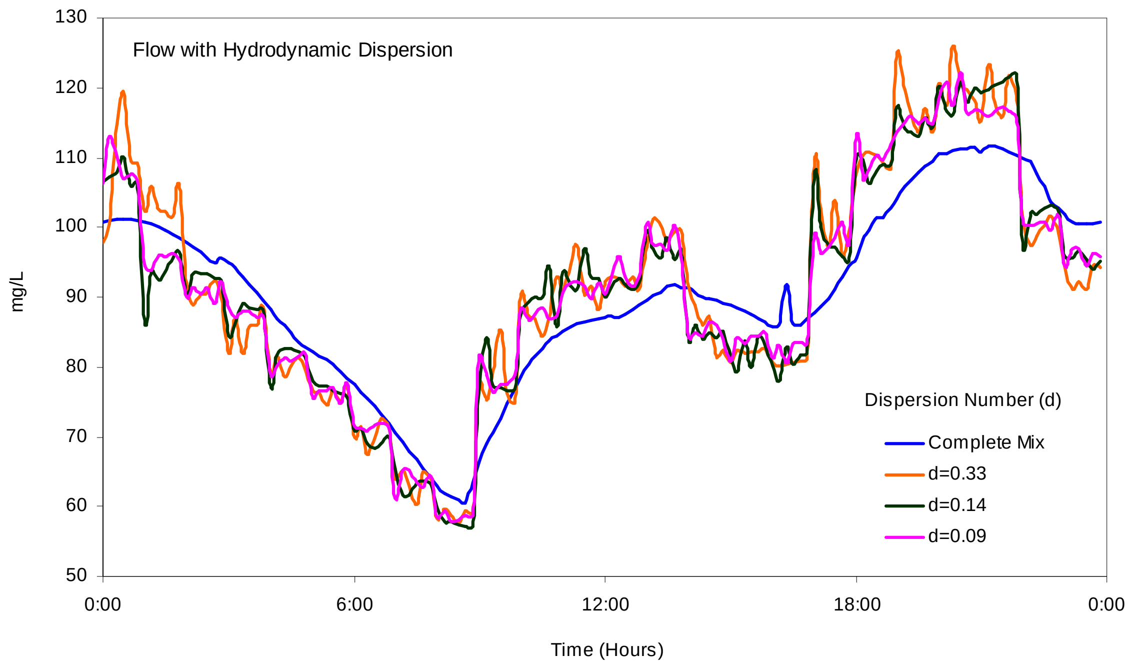
FIGURE 4.6
PRIMARY INFLUENT TSS CONCENTRATIONS WITH AND WITHOUT TLW



4.1.5 Hydraulic Characteristic in the Effluent Channels

In a previous dye test study, results suggested that ideal complete mixing could be achieved in the pre-aeration tanks and ideal plug-flow conditions could be achieved in the primary sedimentation tanks (CH2M Gore and Storrie, 1996). There was no test conducted to examine the hydraulic characteristics of the effluent channel (north and south), however some non-ideal mixing can be expected due to their longitudinal configurations (e.g. the length to width ratios are about 180:1 of the north channel and 120:1 of the south channel). Non-ideal mixing due to longitudinal advection and dispersion may have effects on the effluent concentrations at the end of the channel, particularly to the flow proportional composition samples. A typical example of BOD concentration profile at the end of channel is illustrated in Figure 4.7, simulating various non-ideal mixings in the north and south channels with different dispersion numbers (d). Significant differences and fluctuations were projected with different degrees of mixing as d=0.33, =0.14 and =0.09. (For ideal mixing, d=∞; for ideal plug flow, d=0; where, d is a unitless dispersion factor equal to the axial dispersion coefficient divided by the velocity and a significant dimension of the reactor.)

FIGURE 4.7
BOD EFFLUENT CONCENTRATION PROJECTIONS WITH HYDRODYNAMIC DISPERSION EFFECT



4.1.6 Primary Sedimentation Tank Removal Efficiency and TLW Discharge Scenarios

The primary sedimentation tank removal efficiency is mainly dependent on the particle size and weight distribution, temperature, viscosity, influent flow rate (surface overflow rate) and weir loading. Solids loading flux and operational condition (e.g. underflow rate and duration) may also contribute to the removal efficiencies. The removal efficiencies of BOD, TSS and SBOD in the PST were assumed at 40%, 60% and 0%, respectively (SBOD removal in the PST is negligible). Three cases of TLW discharges were simulated to obtain the effluent composition concentrations:

- Case I: with both non-domestic and domestic TLW
- Case II: with domestic TLW only
- Case III: without non-domestic and domestic TLW

Preliminary investigation suggested that non-ideal mixing in the effluent channel resulted in only marginal increases of effluent concentrations by 0~6 mg/L (flow-rate proportional composite), compared to the complete mixing cases. These differences may be caused by the axial dispersion in the narrow channel and higher sampling frequency during high flow condition (also high concentrations). Since the effect of non-ideal mixing in the effluent channels is considered insignificant, a complete mixing condition is assumed in the modified mass-balance analysis to project the primary effluent quality.

4.2 RESULTS AND SIGNIFICANCE

A modified mass balance was conducted to project the TLW impacts onto the effluent composite concentrations (flow-rate proportional composites). The results of time-series effluent concentrations (instantaneous readings) are included in the Appendix C, and the estimated flow-rate proportional composite concentrations of BOD, SBOD and TSS are summarized in Table 4.1. The values presented in Table 4.1 are intended to be used for comparisons among three different cases, based on various assumptions made in Section 4.1. The results suggested that the domestic and non-domestic TLW discharges caused marginal increases of BOD and SBOD concentrations by 1 to 2 mg/L. The TSS concentration increases caused by domestic and non-domestic TLW discharges were about 2 and 11 mg/L, respectively. The non-domestic TLW seemed to have more significant impact than the domestic TLW on the effluent quality.

The mass balance results suggested that the discharges of TLW contribute a marginal increase of BOD and SBOD in the effluent composite concentrations. The non-domestic TLW could cause a 15% TSS increase in the effluent composite concentration. However, the effluent TSS concentration was not the major concern to meet the current effluent criteria at IIWWTP.

As discussed further in Appendix 10, the main concern at IIWWTP is the immediate concern of non-compliance of the BOD effluent criteria of 130 mg/L. The risk of non-compliance of the TSS effluent criteria of 100 mg/L is not expected to occur until 2010. By that time, it is likely that interim upgrade to deal with the possible BOD exceedances will have been constructed or will be under way. The TLW flow and loads are included in the interim and future treatment capacity upgrade considerations.

Based on the BOD, SBOD and TSS values obtained in this mass balance assessment, we conclude that the addition of TLW to the IIWWTP inflow is not significant to cause effluent non-compliances at the current TLW loading.

The benefit of relocating the non-domestic TLW disposal facility from IIWWTP to AIWWTP cannot be justified, because the effluent quality improvement at IIWWTP is probably limited and an additional facility is needed at AIWWTP.

TABLE 4.1
EFFLUENT CONCENTRATION PROJECTIONS*
(DAILY FLOW-RATE PROPORTIONAL COMPOSITE)

Primary Effluent Composite	BOD, mg/L	SBOD, mg/L	TSS, mg/L
Case I	123	69	76
Case II	121	67	67
Case III	120	67	65

*: Primary removal efficiency: BOD 40%, SBOD 0%, TSS 60%
Case I: with both domestic and non-domestic TLW
Case II: with domestic TLW only
Case III: without both domestic and non-domestic TLW

5 OPERATING STRATEGY

Based on the results of facility assessment, future load projections, service survey, and mass balance exercises, upgrading the existing TLW facility to benefit the interim upgrade goals at IIWWTP cannot be justified. It is further concluded that the GVRD should continue to operate and maintain the existing TLW facilities to their maximum service capacity until secondary treatment is in place. The management of TLW treatment at IIWWTP and AIWWTP should be integrated into the main treatment operation and upgrade considerations (i.e. the interim and build-out to secondary upgrade at IIWWTP). Possible operational improvements, which can be carried out independently or collectively are suggested as follows:

Operation hour expansion

The extension of operation hour at the TLW disposal sites can provide more flexible schedule to the TLW haulers. It may give more convenience to the haulers' operation and allocate the TLW to off-peak hours, which may mitigate the TLW impact on the main treatment process.

Off-peak discharge

Off-peak TLW discharge (e.g. from midnight to dawn) can be arranged by implementing different operation hours or operating a holding tank to pace the TLW discharge, when the main treatment process experiences the lowest flow and loads during the day.

TLW flow distribution

Currently, the pre-treated non-domestic TLW (by rotary drum screen and sedimentation) is discharged directly into the primary sedimentation tanks #11, #12, and #13, which may be overloaded by the high strength of TLW loads. By introducing the treated non-domestic TLW to the influent pump chamber or the grit channel influent, the TLW loads can be evenly distributed into all 15 PST units. This arrangement can be considered in conjunction with the flow distribution improvement at IIWWTP.

Improve the existing pre-treatment facility

Upgrading the existing pre-treatment facility to achieve higher degree of removal efficiency may not be as beneficial and economical, if other operational improvements can better manage the TLW discharge into the main treatment process. However, some facility improvements identified during the site visits can be considered to maximize the treatment capacity and reduce the maintenance requirements:

IIWWTP

- A septic receiving station with pre-screening similar to the AIWWTP can be considered to pre-treat the domestic TLW.
- Re-rate the rotary drum screen capacity and connecting piping in the non-domestic TLW pre-treatment system. Operator has reported that the rotary drum screen could be overloaded during the TLW discharge operation. The bottleneck could be the screen or outlet piping.

AIWWTP

- Replace with a larger grit/stone trap at the bottom of the septic receiving station to reduce the maintenance frequency.

<u>Divert more domestic TLW to AIWWTP for treatment</u>

Since the TLW facility at AIWWTP is designed to receive 50% of the domestic TLW generated in the region, the District should undertake strategies to encourage more domestic TLW to be delivered to AIWWTP for treatment. With the secondary treatment capacity at AIWWTP, probably more than 50% of the domestic TLW can be handled at AIWWTP during the interim period. By diverting more domestic TLW to AIWWTP, the organic and solids loadings to IIWWTP will be correspondingly reduced.

<u>Continue source control and monitoring of the TLW sources</u>

It is important to continue monitor the TLW sources, particularly the non-domestic TLW, to keep tracking of the contaminant loads entering the treatment facility. We have concluded that the non-domestic TLW contaminant loads have increased significantly after examining the sampling results of this study (2003) and the 1997 study (GVRD, 1997). The changes in TLW load characteristics may be caused by many different factors, including the industry operations, source control programs, and bylaw enforcement of pre-treatment. Further TLW source monitoring may also be used for the TLW fee schedule review purpose.

In particular, high soluble BOD liquid waste with low solids contents resulting from spills or process problems at industrial plants should be directed to Annacis Island. This would include airport deicing fluid and contaminated liquid from the beverage and brewing industries.

6 SUMMARY

This trucked liquid waste (TLW) study includes the results of an assessment of the TLW flow and loads generated in the region, and the existing TLW disposal/treatment facility conditions. A service survey of the TLW haulers was conducted in this study to gather information concerning the daily use for current facilities and possible future service changes. A sampling program was carried out in this study to determine the TLW source characteristics, the TLW pre-treatment efficiency, as well as the TLW concentrations entering the main treatment process at IIWWTP. A mass balance analysis was undertaken using the sampling results to evaluate the possible impact of TLW discharge onto the IIWWTP effluent composite concentrations. Results and their significances are summarized as follows:

1. Most of the TLW generated in GVRD is received and treated at either IIWWTP or AIWWTP. Only a small portion of domestic TLW is delivered to the NWLWWTP. Two receiving systems are operated at IIWWTP to handle the domestic and non-domestic TLW separately. The domestic TLW is co-treated with the sewage collected from the Vancouver Sewage Area (VSA), and the non-domestic TLW is processed by a pre-treatment system with rotary drum screen and circular sedimentation tank, prior to entering the rectangular primary sediment tanks of the main wastewater treatment process. A new TLW receiving station and pre-treatment is operated at AIWWTP to handle domestic TLW only. The treated TLW at AIWWTP is directed into the headworks channel and enters the main process for treatment.

2. Currently, AIWWTP receives only approximately 20% of the domestic TLW generated in the region. The rest (80%) is predominately hauled to IIWWTP. The AIWWTP facility was designed to accept 50% of the domestic TLW generated in the region, which suggested that AIWWTP could handle more domestic TLW.

3. The TLW disposal operation hours at IIWWTP are from 5:00 am to 6:00 pm during the weekdays and Saturdays. Haulers are requested to fill the manifests at the attendant office before disposal. An automatic system is operated at AIWWTP with card lock access, video surveillance, and touch-pad human-machine interface (HMI) registration system. The operation hours at AIWWTP has been extended since August 2003, from 5:00 am to 5:00 pm during Monday to Saturday.

4. On average, about 143 truckloads of domestic TLW per month were hauled to AIWWTP. About 703 truckloads of domestic TLW and 120 truckloads of non-domestic TLW were delivered to IIWWTP every month. The average TLW volumes were 9.7 m3/truck of domestic TLW and 6.5 m3/truck of non-domestic TLW.

5. The service survey conducted in this study concluded that there was no serious queuing problem at both TLW disposal sites. Haulers commented to have longer operation hours at the TLW facilities for their convenience, since some of the haulers provide 24h-7d services. The haulers also requested a wash out facility at AIWWTP for their operation conveniences. When asked if Annacis Island was to be the only location for TLW disposal, more than 50% of haulers replied that they were concerned about the traffic condition and hauling distance to AIWWTP.

6. The samples collected in this study were the composite samples collected at the beginning, middle and the end of the TLW discharge. The TLW concentrations sampled in this study were significantly higher than the 1997 numbers by about two folds in every waste category.

7. The removal efficiency of the IIWWTP non-domestic TLW pre-treatment was estimated at about 20 to 80% of BOD, negligible for SBOD, and 30 to 60% of TSS. An average of about 45% TSS removal was estimated based on the weekly mass balance calculations. However, these removal efficiencies were probably the results of primary scum flow dilution, which was introduced into this pre-treatment system when there was no TLW discharge.

8. The results of the mass balance analysis conducted in this study suggested that the TLW discharge could cause TSS concentration increases by 2 to 11 mg/L in the IIWWTP effluent composite samples. The increases caused by the TLW BOD and SBOD were about 1 to 2 mg/L only, which was considered insignificant. The hypothesis that the TLW may be a significant factor resulting in the effluent toxicity test failures (due to BOD and SBOD) is not supported, unless other unknown toxic constituents in the TLW are present. The mass balance results also revealed the possible consequences of eliminating the domestic TLW discharges at IIWWTP, which may result in only marginal effluent quality improvements (i.e. lower BOD and TSS concentrations in the composite samples).

9. For the interim upgrade objective at IIWWTP, results concluded in this study suggest that there is no immediate need to upgrade the existing TLW treatment facility. Operational changes to maximize the use of existing facilities are considered the most appropriate planning strategy during the interim period. Operation and minor facility improvements can be undertaken independently or collectively to mitigate the TLW impacts, by expanding the operating hours, providing off-peak TLW discharge, undertaking proper flow distribution to prevent overloading some of the treatment units, and diverting more domestic TLW to AIWWTP for treatment. It is considered beneficial to divert more domestic TLW from IIWWTP to AIWWTP to take advantage of its secondary treatment capacity. Installing a pre-screening septic receiving station similar to the AIWWTP can be considered to mitigate the clogging problems at IIWWTP domestic TLW discharge.

10. Should the TLW treatment continue at the regional WWTPs, the TLW flow and loads should be included in the design consideration of future system upgrades (e.g. the interim and build-out to secondary upgrades at the IIWWTP). It is recommended to continue the TLW source monitoring to examine the contaminants and their concentrations.

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APPENDIX A: TLW SAMPLING PROGRAM

1.0 INTRODUCTION

This sampling program is developed to characterize the trucked liquid waste (TLW) characteristics of the domestic and non-domestic sources at the Iona Island Wastewater Treatment Plant (IIWWTP). The effluent quality of the TLW pre-treatment units will also be monitored to determine its treatment efficiency. An optional questionnaire is proposed to gather feedbacks from the TLW haulers/carriers for future planning consideration.

2.0 BACKGROUND

The findings of a mass balance exercise have suggested the importance of knowing the TLW characteristics for impact assessment and planning purposes (Workshop Presentation, June 2003). However, since the latest TLW sampling was conducted in 1997 and the TLW characteristics may have varied from different generators for the past years, additional sampling is needed to detail the TLW compositions. Further characterization of the TLW compositions will also benefit the development of management decisions. Some existing TLW characteristics and their significances are summarized as follows:

- The weighted averages of the domestic and non-domestic TLW used for the pricing calculations, before and after 1997, are summarized in Table A1 (GVRD, 1997).

TABLE A1
TLW CHARACTERISTICS OF WEIGHTED AVERAGE CONCENTRATIONS

Parameters	Before 1997		After 1997	
	Domestic	Commercial	Domestic	Commercial
TSS (mg/L)	3,000	74,850	5,060	101,060
BOD (mg/L)	3,000	40,700	1,560	41,250

- The 1997 sampling program (GVRD 1997) reported the soluble BOD (SBOD) to total BOD ratio at an average of 34% (ranged from 7% to 92%).

- The breakdowns by the TLW generation sources adopted in the 1997 sampling program are listed as follows (GVRD 1997):

Domestic TLW	Non-Domestic TLW
1. Septage Tank	1. Beverage
2. Holding Tank	2. Fish Processing
3. Portable Toilets	3. Fruit & Vegetable
4. Others	4. Poultry
	5. Meat
	6. Waste Reduction
	7. Restaurant
	8. Others

- A 2002 summary report documented a one-day sampling event (June 26, 2002) of the non-domestic TLW and pre-treatment effluent. The TLW BOD ranged from 3,630 mg/L to 11,800 mg/L, and the composite samples of pre-treatment effluent concentrations were 1,980 mg/L of BOD and 2,190 mg/L of TSS (GVRD 2002).

- Currently, only about 17% of the domestic TLW generated in the region are hauled to the AIWWTP for disposal since its commissioning in November 2002. The domestic TLW are pre-treated by the semi-cylindrical screening and auger dewatering at the AIWWTP septage receiving station, before entering the plant headwork process.

- Most of the domestic TLW generated in the region are delivered to the IIWWTP and been discharged directly into the influent siphons entering the plant headwork processes for treatment.

- The only non-domestic TLW receiving and pretreatment facilities in the region are located in the IIWWTP. The non-domestic TLW are pre-treated by screening and settling before entering the plant primary treatment system. The performance of the non-domestic TLW pre-treatment has never been rated since its commissioning in 1997.

- The mass balance exercise suggested the significant impacts of non-domestic TLW on the plant treatment and effluent quality, due to its discharge pattern and loads. Additional sampling is required to verify the TLW impacts and pre-treatment performance.

3.0 PROPOSED SAMPLING PROGRAM FOR DETERMINING TLW CHARACTERISTICS AND PRE-TREATMENT EFFICIENCY

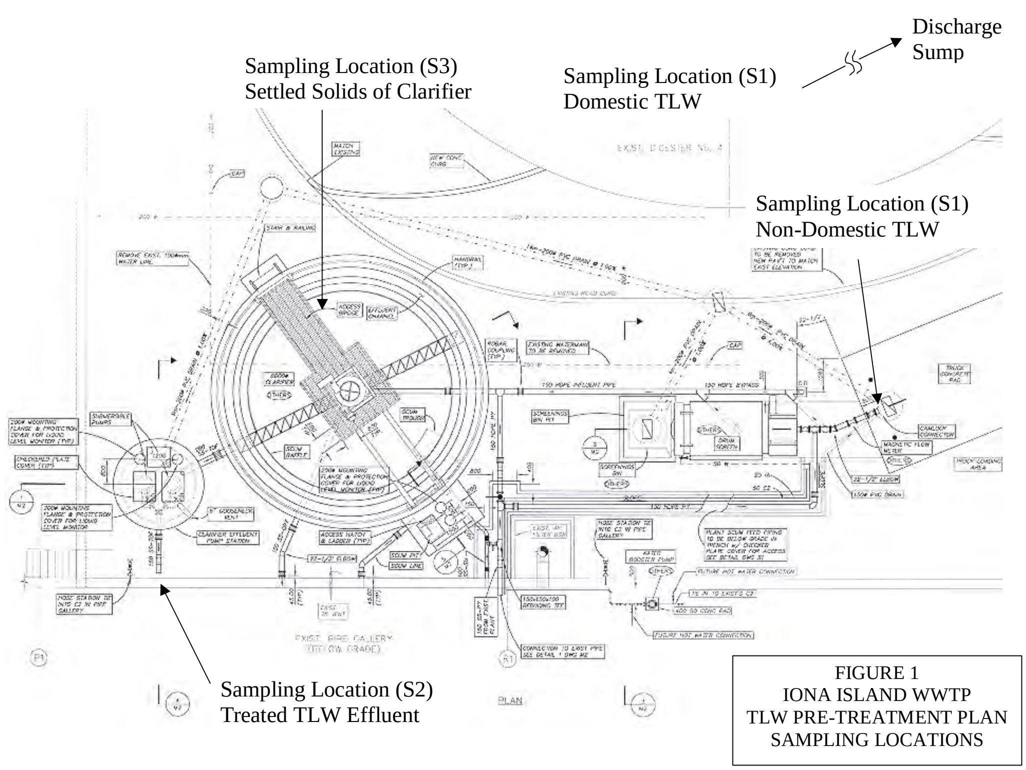
The proposed sampling program is designed to determine the TLW characteristics (domestic and non-domestic sources), and the treatment efficiency of the non-domestic TLW pre-treatment facility at IIWWTP. Grab sampling of a single truckload is unlikely to characterize the TLW load properly, and a specific truckload is unlikely to be representative of a given category of TLW type. A composite sample with discrete manual samples collected during the beginning, middle and end of discharge, is suggested for the TLW sampling. Due to the instantaneous discharge of TLW, time series sampling is considered for the pre-treatment effluent monitoring, to assess the pre-treatment efficiency.

Provisionally, the sample containers (30 plastic bottles and lids, 500 mL each), sample storage (refrigerator) and lab space (for sample preparation and handling) will be provided by GVRD (e.g. IIWWTP lab).

3.1 Sampling Locations

The S1 samples are designed to characterize the domestic and non-domestic TLW compositions and the S2 samples are obtained to evaluate the pre-treatment efficiency. The proposed sampling locations are shown in Figure A1.

FIGURE A1
SAMPLING LOCATIONS



The domestic TLW samples will be collected from the discharge chamber adjacent to the plant influent siphons. The non-domestic TLW samples will be collected at the effluent channel of the rotary drum screen (degritted samples). The pre-treatment effluent samples (S2) will be taken from the pre-treatment effluent pump station. Since the settled solids at the pre-treatment clarifier are withdrawn only once a week, grab samples of the bottom solids (S3) are collected during the withdraw.

- S1: domestic and non-domestic TLW
- S2: pre-treatment effluent
- S3: settled solids at the pre-treatment clarifier

3.2 Sampling Type, Volume and Frequency

The S1 samples will be collected only during the TLW discharge. Composite samples are proposed to collect discrete samples during the beginning, middle, and end of each TLW discharge. The sample volume of each collection event will be 1,000 mL and the total composite sample volume of each truckload will be 3,000 mL. Half liter (500 mL) of the composite sample will be stored in the fridge and shipped to the lab for analysis. The TLW sampling timing should be predetermined by the sampling staff, subject to the TLW discharge rates that may vary from 10 to 30 minutes approximately.

The S2 samples will be collected every two hours discretely, starting from 8:00am to 6:00pm. Half liter (500 mL) of sample will be collected from each sampling event and stored in the fridge for lab analysis. The S3 samples (500 mL each) will be taken from the bottom of the pre-treatment clarifier at the end of draining (scheduled maintenance).

The chain-of-custody log will be recorded for each sample collected.

3.3 Analytical Parameters

Three analytical parameters are proposed to characterize the TLW samples, including the total suspended solids (TSS), biochemical oxygen demand (BOD) and soluble BOD (SBOD). The sample collection, preservation and storage should comply with the procedures and requirements specified in the Standard Method 2540D, 5210, and 5220 (APHA et al. 1995) or equivalent methods. The analytical method should follow the procedures specified in the Standard Method accordingly. The S3 samples will be analyzed for the TSS only.

3.4 Monitoring Duration and Sample Size

The monitoring duration of S1 are divided into two stages. The Phase 2 sampling should be initiated after the analytical results are obtained from the Phase 1. Sampling frequency of Phase 2 will be adjusted if necessary, subject to the Phase 1 results. Preferably, five truckloads of each TLW category (see the TLW category breakdowns in Section 2.0) will be sampled.

- Phase 1: 20 working hours
- Phase 2: 20 ~ 30 working hours (to be determined after receiving the Phase 1 analytical results). More sampling hours can be expended if necessary to collect representative non-domestic TLW samples.

One day sampling will be conducted to collect S2 samples during Phase 1 and Phase 2, respectively.

The total sample sizes are 64 samples, fifty (50) samples of S1, twelve (12) samples of S2, and two (2) samples of S3.

- At S1 location: five (5) samples for each TLW category (10 types $\times$ 5 samples/type = 50 samples in total).
- At S2 location: 6 samples of each sampling day (2 days $\times$ 6 samples/day = 12 samples in total).
- At S3 location: two (2) settled solids samples at the bottom of the pre-treatment clarifier, analyzed for TSS concentrations only.

The samples actually collected in this program are summarized in Table A2, with their sample ID, TLW volume, date/time of sampling.

3.3 Other Information

The TLW registration information collected at the attendant office are required to categorize the TLW samples, include:

- The TLW volumes of sampled truckloads
- TLW types (source of generation)

For determining the efficiency of the pre-treatment facility, the following information are needed for a mass balance exercise:

- Scum pump flow rate (internal recycle from the primary clarifier scum chamber to the pre-treatment clarifier).
- Settled solids volume at the bottom of the pre-treatment clarifier.

5.0 REFERENCES

APHA et al. (1989), Standard Methods for the Examination of Water and Wastewater, 17th edition, American Public Health Association, Washington DC, USA.

GVRD 1997, Trucked Liquid Waste Pricing Strategy Issue Paper

GVRD 2001, Procedure Manual for Use of the Trucked Liquid Waste Facilities.

GVRD 2002, Trucked Liquid Waste Facility Review.

TABLE A2
COLLECTED SAMPLES

Sample ID	TLW Source	Volume, m^3	Time of Sampling
Aug-7-2003			
DOC-001	Portable Toilet	3.6	9:45
DOC-002	Portable Toilet	0.9	10:15
DOC-003	Septic/holding tank	18.2	10:30
DOC-004	Holding tank	9.1	10:50
DOC-005	Septic tank	8.2	11:00
DOC-006	Septic/holding tank	20.5	11:15
DOC-007	Holding tank	4.8	11:30
DOC-008	Septic tank	5.3	11:50
DOC-009	Septic tank	0.7	11:55
DOC-010	Septic tank	0.7	12:10
DOC-011	Portable Toilet	6.0	12:10
DOC-012	Portable Toilet	6.0	12:10
DOC-013	Portable Toilet	3.2	12:10
Non-DOC-001	Restaurant	2.3	8:30
Non-DOC-002	Meat	6.8	10:15
S3-Phase 1	TLW clarifier bottom	-	8:00
DOC-014	Holding tank	15.9	12:55
DOC-015	Fish	27.3	13:30
Aug-8-2003			
S2-001	TLW clarifier effluent	-	6:00
S2-002	TLW clarifier effluent	-	8:00
S2-003	TLW clarifier effluent	-	10:00
S2-004	TLW clarifier effluent	-	12:00
S2-005	TLW clarifier effluent	-	14:00
Non-DOC-003	Waste Reduction	4.6	7:00
Non-DOC-004	Restaurant	2.3	8:30
Non-DOC-005	Waste reduction	4.6	11:00
Non-DOC-006	Waste reduction	4.6	11:15
Non-DOC-007	Restaurant	2.3	13:30
Aug-26-2003			
Non-DOC-004	Waste Rreduction	6.8	11:30
Non-DOC-005	Poultry	4.6	15:20
Aug-28-2003			
Non-DOC-001	Waste Reduction	4.6	7:00
S2-001	TLW clarifier effluent	-	8:00
S2-002	TLW clarifier effluent	-	9:00
Non-DOC-003	Restaurant	2.1	9:45
S2-003	TLW clarifier effluent	-	10:00
Non-DOC-004	Waste Reduction	4.6	10:30
S2-004	TLW clarifier effluent	-	11:00
S2-005	TLW clarifier effluent	-	12:00
S2-006	TLW clarifier effluent	-	13:00
S2-007	TLW clarifier effluent	-	14:00
S2-008	TLW clarifier effluent	-	15:00
S2-009	TLW clarifier effluent	-	16:00
Non-DOC-007	Poultry	6.8	13:30

APPENDIX B: TLW SERVICE SURVEY FORM

GREATER VANCOUVER REGIONAL DISTRICT
Trucked Liquid Waste (TLW)
Questionnaire
2003 August

Please check the box ? ☐ Please check the box ? ☐

1. **Is the current TLW operation hours suitable to your hauling operation?**
   - Iona Island YES ☐ NO ☐ If No, your preferable hours___________
   - Annacis Island YES ☐ NO ☐ If No, your preferable hours___________
2. **Have you experienced waiting for discharge more than 10 minutes at a single trip?**
   - Iona Island Never ☐ Occasional ☐ Very often ☐
   - Annacis Island Never ☐ Occasional ☐ Very often ☐
3. **What type of waste do you haul/carry at this trip?**
   - Domestic waste (septage, portable toilet, pump station, holding tank) ☐
   - Non-Domestic waste (none of the category listed above) ☐
4. **If only the Annacis Island is open for the *DOMESTIC* disposal, will it affect your operation?**
   - NO ☐ YES ☐ if YES, why? distance ☐ traffic condition ☐ hours ☐ other __________
5. **If only the Annacis Island is open for the *NON-DOMESTIC* disposal, will it affect your operation?**
   - NO ☐ YES ☐ if YES, why? distance ☐ traffic condition ☐ hours ☐ other __________

Please indicate if you have any specific comment regarding the TLW receiving service in GVRD?

____

____

We appreciate your participation. Your feedbacks are important to us for future service improvements.
Please fill in your information to enter a **$100 gift certificate** draw
(one person per entry)

Your Name: ____

Company Name: ____

Contact Phone Number:______________________________________

Signature: ____

APPENDIX C: TLW IMPACT MASS BALANCE – ESTIMATED EFFLUENT CONCENTRATIONS OF BOD, SBOD, AND TSS

	Case I: Tank #11~ #13 with non-domestic TLW			**Case II:** Other tanks without non-domestic TLW			**Case III:** All tanks without domestic and non-domestic TLW		
Sampling Time	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L
0:00	105	67	62	105	67	62	105	67	62
0:15	103	65	62	103	65	62	103	65	62
0:30	98	62	61	98	61	61	98	61	61
0:45	95	59	60	95	58	60	95	58	60
1:00	92	56	59	92	56	59	92	56	59
1:15	88	53	59	88	53	58	88	53	58
1:30	85	50	58	85	50	58	85	50	58
1:45	82	47	57	81	47	57	81	47	57
2:00	78	45	56	78	44	56	78	44	56
2:15	76	43	55	76	43	54	76	43	54
2:30	74	42	53	74	41	53	74	41	53
2:45	72	40	52	72	40	51	72	40	51
3:00	71	39	50	70	38	50	70	38	50
3:20	69	37	48	68	37	48	68	37	48
3:40	65	35	46	65	34	45	65	34	45
4:00	63	33	44	62	32	43	62	32	43
4:20	60	31	42	59	30	41	59	30	41
4:40	58	29	40	57	28	39	57	28	39
5:00	55	27	37	54	26	37	54	26	36
5:20	52	25	35	52	24	34	51	24	31
5:40	50	23	33	49	22	32	47	21	26
6:00	48	22	31	46	20	30	44	19	21
6:20	52	24	33	50	22	32	48	20	23
6:40	57	26	35	54	23	34	51	22	24
7:00	61	29	38	57	25	37	55	23	26
7:20	69	31	59	61	26	39	59	25	29
7:40	76	34	79	65	27	41	63	26	32
8:00	82	36	96	68	29	43	67	28	35
8:15	83	36	82	71	30	45	69	29	37
8:30	83	36	70	74	31	46	72	30	39
8:45	84	36	59	77	32	48	75	32	41
9:00	85	36	50	79	34	50	78	33	43
9:10	87	37	58	81	34	51	80	33	44
9:20	89	38	61	82	35	51	81	34	44
9:30	91	39	68	84	35	52	82	35	45
9:40	94	40	75	86	36	53	84	35	46
9:50	96	41	81	88	37	54	86	36	47
10:00	99	42	87	90	38	55	88	37	48
10:05	100	43	91	91	38	56	89	37	49
10:10	101	44	86	93	40	57	91	39	50
10:15	102	44	84	94	41	58	93	40	51
10:20	102	45	82	96	42	59	94	41	52
10:25	103	46	79	97	43	59	95	42	53
10:30	103	46	77	98	44	60	97	43	54
10:35	104	47	75	100	45	61	98	44	55
10:40	105	47	73	101	45	61	99	45	56
10:45	105	48	71	102	46	62	101	46	56
10:50	106	49	69	103	47	63	102	47	57
10:55	106	49	67	105	48	64	103	48	58

	Case I: Tank #11~ #13 with non-domestic TLW			Case II: Other tanks without non-domestic TLW			Case III: All tanks without domestic and non-domestic TLW		
Sampling Time	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L
11:00	107	50	65	106	49	64	105	49	59
11:05	109	51	69	107	50	65	106	50	60
11:10	110	53	73	108	51	66	107	50	60
11:15	112	54	76	110	52	66	108	51	61
11:20	114	55	80	111	53	67	110	52	62
11:25	116	56	84	112	54	68	111	53	62
11:30	117	58	87	114	55	68	112	54	63
11:35	119	59	91	115	56	69	114	55	64
11:40	121	60	95	116	57	70	115	56	65
11:45	122	62	98	117	58	70	116	57	65
11:50	124	63	102	119	59	71	117	58	66
11:55	126	64	105	120	60	72	119	59	67
12:00	127	65	109	121	61	72	120	60	67
12:05	128	66	106	122	61	73	121	61	68
12:10	129	67	104	124	62	74	123	62	69
12:15	130	67	102	125	63	74	124	63	70
12:20	131	68	99	126	64	75	125	64	70
12:25	132	68	97	128	65	76	126	65	71
12:30	132	69	95	129	66	76	128	66	72
12:35	133	69	92	130	67	77	129	67	73
12:40	134	70	90	131	68	78	130	67	73
12:45	135	71	88	133	69	79	132	68	74
12:50	136	71	86	134	70	79	133	69	75
12:55	137	72	83	135	71	80	134	70	76
13:00	137	73	81	137	72	81	135	71	76
13:05	139	74	85	138	73	81	137	72	77
13:10	141	75	88	139	74	82	138	73	78
13:15	143	76	92	140	75	83	139	74	79
13:20	145	77	95	142	76	83	141	75	80
13:25	147	78	99	143	76	84	142	76	80
13:30	148	80	103	144	77	85	143	77	81
13:35	150	81	107	145	78	85	145	78	82
13:40	152	82	111	147	79	86	146	79	83
13:45	154	83	114	148	80	87	147	80	83
13:50	156	84	117	149	81	87	148	81	84
13:55	158	86	121	151	82	88	150	82	85
14:00	159	87	124	152	83	89	151	83	86
14:05	159	87	121	152	83	88	151	83	86
14:10	159	87	118	152	83	88	151	83	85
14:15	158	87	115	152	84	88	151	83	85
14:20	158	87	111	152	84	87	152	83	85
14:25	157	86	108	152	84	87	152	84	84
14:30	157	86	105	152	84	86	152	84	84
14:35	157	86	102	153	84	86	152	84	84
14:40	156	86	98	153	85	86	152	84	83
14:45	156	86	95	153	85	85	152	84	83
14:50	156	86	92	153	85	85	152	85	82
14:55	155	86	88	153	85	84	152	85	82
15:00	155	86	85	153	85	84	153	85	82
15:05	155	86	84	153	85	84	153	85	81
15:10	155	86	84	153	86	83	153	85	81
15:15	155	87	83	153	86	83	153	86	81
15:20	155	87	83	154	86	83	153	86	81
15:25	155	87	83	154	86	82	153	86	80
15:30	155	87	82	154	86	82	153	86	80
15:35	155	87	82	154	87	81	154	86	80
15:40	155	87	81	154	87	81	154	87	79
15:45	155	87	81	154	87	81	154	87	79
15:50	155	88	80	154	87	80	154	87	79
15:55	155	88	80	154	87	80	154	87	79

	Case I: Tank #11~ #13 with non-domestic TLW			**Case II:** Other tanks without non-domestic TLW			**Case III:** All tanks without domestic and non-domestic TLW		
Sampling Time	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L	BOD conc mg/L	SBOD conc mg/L	TSS conc mg/L
16:00	155	88	80	155	88	79	154	87	78
16:10	155	88	79	155	88	79	155	88	78
16:20	155	88	78	155	88	78	155	88	77
16:30	155	89	78	155	88	77	155	88	77
16:40	155	89	77	155	89	77	155	89	76
16:50	155	90	76	156	89	76	155	89	76
17:00	156	90	75	156	90	75	156	90	75
17:10	156	90	75	156	90	74	156	90	74
17:20	156	91	74	156	90	74	156	90	73
17:30	156	91	73	156	91	73	156	91	73
17:40	156	91	72	157	91	72	157	91	72
17:50	156	92	71	157	91	71	157	91	71
18:00	157	92	71	157	92	70	157	92	70
18:10	156	92	70	157	92	70	157	92	70
18:20	155	92	70	156	92	70	156	92	70
18:30	154	92	70	155	92	70	155	92	70
18:40	153	92	70	154	92	70	154	92	70
18:50	152	92	70	152	92	70	152	92	70
19:00	151	92	70	151	92	70	151	92	70
19:10	150	92	70	150	91	69	150	91	69
19:20	149	91	70	149	91	69	149	91	69
19:30	148	91	70	148	91	69	148	91	69
19:40	147	91	69	147	91	69	147	91	69
19:50	146	91	69	146	91	69	146	91	69
20:00	145	91	69	145	91	69	145	91	69
20:10	144	91	69	144	91	69	144	91	69
20:20	143	91	69	143	91	69	143	91	69
20:30	142	91	69	142	91	69	142	91	69
20:40	141	91	69	141	91	69	141	91	69
20:50	140	91	69	140	91	69	140	91	69
21:00	139	91	69	139	91	69	139	91	69
21:10	138	91	69	138	90	68	138	90	68
21:20	136	90	69	137	90	68	137	90	68
21:30	135	90	69	136	90	68	136	90	68
21:40	134	90	68	135	90	68	135	90	68
21:50	133	90	68	134	90	68	134	90	68
22:00	132	90	68	133	90	68	133	90	68
22:10	131	89	68	131	89	68	131	89	68
22:20	128	87	67	129	87	67	129	87	67
22:30	126	85	67	126	85	67	126	85	67
22:40	124	83	67	124	83	66	124	83	66
22:50	122	82	66	122	81	66	122	81	66
23:00	120	80	66	120	79	65	120	79	65
23:10	117	78	65	117	78	65	117	78	65
23:20	115	76	65	115	76	64	115	76	64
23:30	113	74	64	113	74	64	113	74	64
23:40	111	72	64	111	72	63	111	72	63
23:50	108	70	63	108	70	63	108	70	63
Composite	123	69	76	121	67	67	120	67	65

APPENDIX D: RESULTS FROM SAMPLE ANALYSIS

B.C. Analytical Technologies Ltd.
BCAT
120A - 3989,Henning Drive, Burnaby, BC
V5C 6N5 Canada
Phone: 604-320-1588 Fax: 604-434-9577
E-mail: pkshieh@bcat.ca

Fax

To:	Stantec	**Attn:**	Dr. Jowitt Li
Fax:	604-591-1856	**Pages:**	5 (include this cover Page)
From:	Parker Shieh	**Date:**	Aug 14, 2003
Re:	Preliminary analytical report	**CC:**	

☐ **Urgent** ☐ **For Review** ☐ **Please Comment** ☐ **Please Reply** ☐ **Please Recycle**

☐● **Comments:**

Hi Jowitt,

Please find the preliminary report for the first batch of samples. Final report with QA/QC will be followed as soon as it is certified. Please feel free to contact me should you have any questions.

Regards,

Parker

BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #:	A0308003S47	*Project:*	117-0001801
FOR:	Stantec Consulting Suite 1007, #7445, 132Street Surey, B.C. V3W 1J8	*Date Received:*	August 7, 2003
		Date Completed:	August 14, 2003
		Fax #:	604-591-1856
		Phone #:	604-597-0442
Attn:	Jowitt Li	*Account:*	HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
DOC-001	Aug-7-03	Water	3700	1200	1120
DOC-002	Aug-7-03	Water	8100	1300	57400
DOC-003	Aug-7-03	Water	930	95	11200
DOC-004	Aug-7-03	Water	290	88	701
DOC-005	Aug-7-03	Water	8700	2400	24200
DOC-006	Aug-7-03	Water	460	130	409
DOC-007	Aug-7-03	Water	450	110	2270
DOC-008	Aug-7-03	Water	830	320	349
DOC-009	Aug-7-03	Water	1100	170	72000
DOC-010	Aug-7-03	Water	17000	2700	77600

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BCAT British Columbia Analytical Technologies Ltd.

Analytical Results:

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
DOC-011	Aug-7-03	Water	2800	1100	34100
DOC-012	Aug-7-03	Water	2700	830	7450
DOC-013	Aug-7-03	Water	9100	3400	143000
NON-DOC-001	Aug-7-03	Water	85000	3700	127000
NON-DOC-002	Aug-7-03	Water	25000	2300	95500
S3-Phase 1	Aug-7-03	Water	-	-	164000
DOC-014	Aug-7-03	Water	730	330	920
DOC-015	Aug-7-03	Water	7400	3600	18100

Analytical Results Certified by: ______________________________

Dr. Parker Shieh: Director, Environmental Services

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Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Total BOD-5day	Incubation/DO APHA5210 B 4500-OGC	5	101
Soluble BOD-5day	Filtration/Incubation/DO APHA5210 B 4500-OGC	5	102
Total Suspended Solids	Filtration/Gravimetric APHA2540D	1	104

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Biochemical Oxygen Demand

A series of appropriate dilutions of the sample is prepared with dilution water (containing buffer, nutrients and seed solution) in a BOD bottle. Dissolved oxygen is measured 15 minutes after preparation. The solution is incubated for 5 days at 20 °C in the dark and dissolved oxygen is again measured (APHA 4500-OG, Membrane Electrode and APHA 4500-OC, Modified Winkler). The BOD content is computed form the difference between the two measurements.

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Total Suspended Solids

A well-mixed sample aliquot (100 mL) is filtered through a glass fiber filter (Whatman 934-AH), and the residue retained on the filter is dried to constant weight at 103-105 deg C. The filter is then desiccated for final weight measurement. The TSS is determined gravimetrically by the difference of filter before and after the filtration.

Soluble Biochemical Oxygen Demand

The sample is filtered through filtration system with a 0.45 um membrane. The filtrate, defined as a soluble solution, is then processed according to the analytical method of APHA 4500-OG for the determination of 5-day Biochemical Oxygen Demand.

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BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #:	A0308005S49	*Project:*	117-0001801
FOR:	Stantec Consulting	*Date Received:*	August 8, 2003
	Suite 1007, #7445, 132Street	*Date Completed:*	August 15, 2003
	Surey, B.C.	*Fax #:*	604-591-1856
	V3W 1J8	*Phone #:*	604-597-0442
Attn:	Jowitt Li	*Account:*	HOY

Analytical Results: (DRAFT)

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
S2-001	Aug-8-03	Water	360	44	296
S2-002	Aug-8-03	Water	220	74	10,300
S2-003	Aug-8-03	Water	110	59	426
S2-004	Aug-8-03	Water	120	37	12,500
S2-005	Aug-8-03	Water	180	48	247
NON DOC-003	Aug-8-03	Water	>40,000	17,000	278,000
NON DOC-004	Aug-8-03	Water	16,000	>4,000	41,600
NON DOC-005	Aug-8-03	Water	>40,000	>4,000	127,000
NON DOC-006	Aug-8-03	Water	>40,000	33,000	552,000
NON DOC-007	Aug-8-03	Water	>40,000	>4,000	148,000

SEP 05 2003

BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308016S60

FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8

Attn: Jowitt Li

Project: NA
Date Received: August 26, 2003
Date Completed: September 4, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
NON-DOC-004 Ph 2	Aug-22-03	Water	500,000	8,000	700,000
NON-DOC-005 Ph2	Aug-22-03	Water	110,000	15,000	30,000

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BCAT British Columbia Analytical Technologies Ltd.

Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Total BOD-5day	Incubation/DO APHA5210 B 4500-OGC	5	101
Soluble BOD-5day	Filtration/Incubation/DO APHA5210 B 4500-OGC	5	102
Total Suspended Solids	Filtration/Gravimetric APHA2540D	5	104

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

Analytical Results Certified by: ______________________________

Dr. Parker Shieh: Director, Environmental Services

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SEP 0 5 2003

BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308018S62

FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8

Attn: Jowitt Li

Project: NA
Date Received: August 28, 2003
Date Completed: September 3, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
NON-DOC-001 PH3*	Aug-28-03	Water	180,000	23,000	1,330,000
S1-001 PH3	Aug-28-03	Water	3,400	470	2,180
S2-002 PH3	Aug-28-03	Water	960	190	634
NON-DOC-003 PH3	Aug-28-03	Water	77,000	4,900	119,000
S2-003 PH3	Aug-28-03	Water	1,200	100	1,600
NON-DOC-004 PH3*	Aug-28-03	Water	150,000	110,000	900,000
S2-004 PH3	Aug-28-03	Water	6,300	190	4,700
S2-005 PH3	Aug-28-03	Water	2,400	650	8,440
S2-006 PH3	Aug-28-03	Water	2,300	500	877
S2-007 PH3	Aug-28-03	Water	4,500	480	3,900

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BCAT British Columbia Analytical Technologies Ltd.

Client Sample Identification	Sample Date	Matrix	BOD, 5-Day (mg/L)	SBOD 5-Day (mg/L)	Total Suspended Solids (mg/L)
S2-008 PH3	Aug-28-03	Water	1,700	240	1,160
S2-009 PH3	Aug-28-03	Water	1,100	320	769
NON-DOC-007 PH3	Aug-28-03	Water	170,000	16,000	226,000

* Sample "NON-DOC-001 Ph3" and "NON-DOC-004 Ph3" contain large amounts of oil and grease.

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GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 2
Low DO in Iona WWTP Tributary Network

FINAL REPORT

Prepared for

Greater Vancouver Regional District

Prepared by

Stantec Consulting Ltd.
#1007, 7445 – 132nd Street
Surrey, BC
V3W 1J8

Dayton & Knight Ltd.
#210, 889 Harbourside Drive
North Vancouver, BC
V7P 3S1

Contact: Mr. Gilbert Cote, P.Eng
(Stantec Consulting Ltd.)
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Fax: (604) 591-1856

August 2005

117-00018

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1 INTRODUCTION

2.2 1.1 PROJECT OVERVIEW

Stantec Consulting Ltd., in association with Dayton & Knight Ltd., has been retained by GVRD to provide professional engineering consulting services for the development of Facility Plans for the Iona Island and Lions Gate Wastewater Treatment Plants.

This Appendix detail the work carried to deal with the low dissolved oxygen in the Iona Island Wastewater Treatment Plant (IWWTP) tributary network.

1.2 SCOPE OF WORK

The scope of work for this portion of the project as outlined in the GVRD terms of reference is summarized briefly as follows:

i) A review of the existing sewerage system tributary to the Iona Island WWTP with respect to low dissolved oxygen (DO) levels and its impact on effluent toxicity. The objective of this work is to calibrate the DO profile of the plant influent for the low flow (summer) dry weather conditions.

   As discussed at the sewer system modeling workshops, the real impact on primary effluent toxicity is not the level of the dissolved oxygen in the effluent but the concentration of readily degradable organic material in the wastewater and primary effluent, which during the batch toxicity tests causes an oxygen utilization rate, which exceeds the allowable re-aeration rates during the test. During the 96 hour test period, the organisms present in the primary effluent utilize the oxygen in the samples at such a rate that the dissolved oxygen levels drop to levels at which fish mortality occurs through oxygen starvation. Sometimes this occurs within the first few hours of the test. Stantec's monitoring and modeling work therefore will concentrate on the modeling of the BOD and COD levels in the influent and primary effluent.

ii) Review Key Manhole Monitoring Programs completed to date.

iii) Develop 2003 Monitoring Program location, frequency and water quality parameter requirements.

iv) Review and analyze 2003 Monitoring Results.

v) Provide direction to the Greater Vancouver Sewerage & Drainage District (GVRD) staff in model calibration efforts.

vi) Review and analyze modeling efforts completed by GVRD.

vii) Identify options to improve the DO level throughout the system.

2 BACKGROUND DATA ANALYSIS AND REVIEW

2.1 DESCRIPTION OF STUDY AREA

2.2.1 General

The major interceptor sewers to be modeled as part of this assignment include the following:

i) 8th Avenue Interceptor (8AI) (Cambie Street west to Highbury Street).

ii) Highbury Interceptor (HI) (8th Avenue south to the Iona IIWWTP).

iii) North Arm Interceptor (NAI) (Barnard Street west to Highbury Street).

Figure 2.1 provides a partial Key Plan for the Iona Island Tributary Network. A brief overview of each major interceptor sewer follows.

2.2.1 8th Avenue Interceptor

The 8th Avenue Interceptor sewer drains west from Clark Drive to Highbury Street along 8th Avenue. This interceptor sewer contains three distinct pipe cross-sections, all of relatively flat and varying grades:

i) Clark Drive west to Cambie Street; 2007 mm dia interceptor sewer, slope ranges from 0.047 to 0.088%.

ii) Cambie Street west to Balaclava Street; 2400 mm dia interceptor sewer, slope ranges from 0.109 to 0.131%.

iii) Balaclava Street west to Highbury Street; 2616 mm dia interceptor sewer, slope = 0.166%.

The limits of this study include the trunk sewer located just west of Cambie Street (Ash Street) west to Highbury Street. This includes an 1829 mm dia sewer connection at Balaclava Street.

FIGURE 2.1

KEY PLAN

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DRAWING SYMBOLS

PIPELINE, TRAVERSE HUB, MONUMENT, IRON PIN FOUND, OLD IRON PIN, LEAD PLUG FOUND, MANHOLE, POWER POLE, TELEPHONE POLE, GUY/ANCHOR POLE, WATER VALVE BOX, GAS VALVE BOX, SEWER HOUSE CONNECTIONS, FIRE HYDRANT, SEWER, STORM DRAIN, CATCH BASIN, CULVERT, GAS MAIN, STORM HOUSE CONNECTION, UNDERGROUND TEL. CABLES, WATER MAIN, WATER HOUSE CONNECTION, RAILWAY SIGNAL LIGHT, HOUSE NUMBER, UG ELECT. CABLES, LAMP STANDARD, DITCH, PAVEMENT, ROCKERIES, FENCE, GRAVEL SURFACE, ELECT. PULL BOX, SIGN, HEDGE, TREE, CAPPED SERVICE PIPE, PROPOSED PERMANENT EASEMENT, EXISTING PROPERTY LINE, TEMP. WORKING SPACE, AIR VALVE, GRADE BREAK, BLOWDOWN

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	Appd	Description
P1	04/24/03	GKR	BMN			ISSUED FOR REVIEW

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: IB/GKR
Drawn: MC/BMN
Checked:
Approved
Manager

FACILITY PLANS FOR IONA ISLAND AND LIONS GATE WWT PLANTS
TASK 2: LOW DO IN IONA

TRIBUTARY NETWORK

SCALE: 1:20000
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:
SAROS DWG. ID No.:

Xrefs: vaawestonno.DWG; facilitygplan0526.DWG;
Drawing: V:\117\ACTIVE\11700018\DRAWINGS\11700018_P01.DWG
December 16, 2003 11:24 a.m.

2.2.1 Highbury Interceptor

i) The Highbury Interceptor sewer collects flows from the 8th Avenue Interceptor sewer and from areas north of the 8th Avenue Interceptor and drains south from 8th Avenue across the North Arm of the Fraser River to the Iona WWTP. This section of sewer consists of both a circular and Boston Horseshoe pipe cross-section, as follows:

ii) 8th Avenue south to 18th Avenue (Jericho Section); 2896 mm dia. interceptor sewer (Boston Horseshoe Cross-Section), slope = 0.058%.

iii) 18th Avenue south to 340 l.m. north of Marine Drive (Tunnel Section); 2946 mm dia interceptor sewer (circular pipe), slope = 0.058%.

iv) 340 l.m. north of Marine Drive to North Arm Interceptor Sewer (Musqueam Section); 2896 mm dia trunk sewer (Boston Horseshoe Cross-Section), slope ranges from 0.0% to 0.14%.

v) North Arm Interceptor Sewer to North Arm Fraser River Crossing (Musqueam Section); 2896 mm dia trunk sewer (Boston Horseshoe Cross-Section), slope = 0.14%.

vi) Fraser River Crossing, 3 – 1727 mm dia interceptor sewers (inverted siphon).

vii) Fraser River Crossing to Iona WWTP, 3 – 1676 mm dia interceptor sewers.

2.2.1 North Arm Interceptor

i) The North Arm Interceptor drains westward from Barnard Street to the Highbury Interceptor Sewer. This interceptor sewer includes a number of distinct pipe / conduit cross-sections as follows:

ii) Barnard Street Section (Barnard Street to Angus Drive); 2743 x 1397 Box (width x height), slope = 0.06%.

iii) Angus Drive Section (Angus Drive to McDonald Street); twin 2134 x 1397 Box (width x height), slope = 0.04%.

iv) McDonald Street Section (McDonald Street to Blenheim Street); twin 2134 x 1397 Box (width x height), slope = 0.04%. This section has the same cross-section as the Angus Drive section but is shown separately because it is on different street.

v) McDonald Street Section (Blenheim Street to the Highbury Interceptor); 3124 x 1829 Box, slope = 0.04%.

2.2 DRY WEATHER FLOW

The GVRD provided the Consultant Team with some typical dry weather flows. This is based on the July 11th – 17th period in 1999. The following typical patterns have been observed:

- Review the existing facilities relative to their location, operation and effectiveness
- Major Low Flow (5:00 to 6:00 a.m.)
- Minor Low Flow (1.65 x Major Low Flow) (4:00 to 5:00 p.m.)
- Major Peak Flow (1.85 x Major Low Flow) (12:00 noon)
- Minor Peak Flow (1.75 x Major Low Flow) (8:00 p.m.)

Table 2.1 provides a summary of minimum / maximum flows and velocities in the three (3) interceptor sewers. This summary was based on data supplied by the GVRD. Information for July 12 was selected as being typical for the July 11 – 17 timeframe.

TABLE 2.1
DRY WEATHER FLOW SUMMARY
JULY 12, 1999

Pipe No. : Node From / To	Typical Dry Weather Flow (cms)		Typical Dry Weather Velocity (m / s)		Low Flow as a % of Low Flow at Iona WWTP	
	Min.	Max.	Min.	Max.	Min.	Max.
86013 - Inlet Node at Iona WWTP	3.92	6.66	n / a	n / a	n / a	n / a
8 th Avenue Interceptor						
85293 (85293 - 85292)	1.65 @ 6:19 am	3.28 @ 10:52 am	1.36 @ 6:19 am	1.63 @ 10:52 am	42.1%	49.2%
85280 (85280 - 85279)	1.67 @ 6:39 am	3.27 @ 10:47 am	1.31 @ 6:39 am	1.60 @ 10:47 am	42.6%	49.1%
85272 (85272 - 85270)	2.08 @ 6:30 am	3.88 @ 11:08 am	1.41 @ 6:30 am	1.67 @ 11:08 am	53.1%	58.3%
85276INFLOW1 (8527Inflow - 85276)	0.34 @ 5:19 am	0.61 @ 10:47 am	n / a	n / a	8.7%	9.2%
Highbury Interceptor						
86044 (86041Inflow - 86041)	1.08 @ 4:45 am	1.66 @ 14:25 am	n / a	n / a	27.6%	24.9%
86039 (86037 - 86038)	3.17 @ 6:30 am	5.44 @ 11:23 am	1.28 @ 6:30 am	1.48 @ 11:23 am	80.9%	81.7%
86036 (86036 - 86034)	3.18 @ 6:35 am	5.44 @ 11:36 am	1.52 @ 6:35 am	1.81 @ 11:36 am	81.1%	81.7%
86032 (86030 - 86028)	3.18 @ 6:43 am	5.44 @ 11:40 am	0.56 @ 6:43 am	0.96 @ 11:40 am	81.1%	81.7%
86031 (86013 to 86007 / 8 / 9)	3.92 @ 6:44 am	6.66 @ 12:28 pm	0.65 @ 6:44 am	0.86 @122:28 pm	100.0%	100.0%
86021 (86021 - 86016)	3.92 @ 6:44 am	6.66 @ 12:28 pm	1.28 @ 6:44 am	1.48 @ 12:28 pm	100.0%	100.0%
86028Inflow (86028Inflow - 86028)	0.02 @ 4:22 am	0.04 @ 18:57 pm	n / a	n / a	0.5%	0.6%
North Arm Interceptor						
85736Inflow (85736Inflow - 85736)	0.07 @ 4:20 am	0.17 @ 8:30 am	n / a	n / a	1.8%	2.6%
85728Inflow1 (85728Inflow1 - 85728)	0.01 @ 4:20 am	0.02 @ 8:30 am	n / a	n / a	0.3%	0.3%
85720Inflow (85720Inflow - 85720)	0.04 @ 4:20 am	0.09 @ 8:30 am	n / a	n / a	1.0%	1.4%
85709Inflow1 (85709Inflow - 85706)	0.02 @ 4:20 am	0.03 @ 8:30 am	n / a	n / a	0.5%	0.5%
85704Inflow (85704Inflow - 85704)	0.03 @ 4:20 am	0.06 @ 8:30 am	n / a	n / a	0.8%	0.9%
85745 (85746 - 85744)	0.54 @ 4:20 am	0.98 @ 12:45 pm	0.71 @ 4:20 am	0.86 @ 12:45 pm	13.8%	14.7%
85731 (85732 - 85730)	0.62 @ 4:20 am	1.11 @ 12:55 pm	0.55 @ 4:45 am	0.67 @ 12:55 pm	15.8%	16.7%
85724 (85725 - 85723)	0.63 @ 4:20 am	1.12 @ 13:10 pm	0.32 @ 4:45 am	0.50 @ 13:10 pm	16.1%	16.8%
85714 (85714 - 85713)	0.71 @ 6:15 am	1.19 @ 13:20 pm	0.32 @ 6:15 am	0.30 @ 13:20 pm	18.1%	17.9%
85705 (85705 - 85704)	0.73 @ 6:15 am	1.22 @ 13:35 pm	0.31 @ 6:15 am	0.34 @ 13:35 pm	18.6%	18.3%
85693 (85693 - 85691)	0.71 @ 6:15 am	1.28 @ 13:35 pm	0.28 @ 6:15 am	0.33 @ 13:35 pm	18.1%	19.2%

2.2.1 8th Avenue Interceptor

i) Low flows typically occur between 6:19 and 6:39 a.m.
ii) Peak flows typically occur between 10:47 and 11:08 a.m.
iii) Peak flows within the 8th Avenue Interceptor are approximately 58% of the peak flows received at the Iona WWTP. Please note this includes the inflow from the 1829 mm dia sewer at Balaclava.
iv) The peak inflow from the Balaclava sewer is approximately 9% of the peak flows received at the Iona WWTP.
v) Velocities in the 8th Avenue Interceptor sewer range from 1.3 – 1.7 m/s.

2.2.2 Highbury Interceptor

i) Low flows typically occur between 6:30 and 6:44 a.m.

ii) Peak flows typically occur between 11:23 a.m. and 12:28 p.m.

iii) Peak flows within the Highbury Interceptor (8th Avenue south to 49th Avenue) are approximately 82% of the peak flows received at the Iona WWTP.

iv) Peak flows from the area south of 8th Avenue into the Highbury Interceptor are approximately 25% of the peak flows received at the Iona WWTP.

v) Velocities in the Highbury Interceptor sewer range from 0.6 – 1.8 m/s.

2.2.3 North Arm Interceptor

i) Low flows typically occur between 4:20 and 4:45 a.m. for sewers located east of McDonald Street to Barnard Street, and around 6:15 a.m. for areas west of McDonald Street to the Highbury Street Interceptor.

ii) Peak flows typically occur between 12:45 and 13:35 p.m.

iii) Peak flows within the North Arm Interceptor are approximately 15-19% of the peak flows received at the Iona WWTP.

iv) Velocities within the North Arm Interceptor range from 0.3 – 0.9 m/s. Velocities for the area west of Yew Street are typically less than 0.5 m/s, even under peak flow conditions.

v) The North Arm Interceptor (downstream of the Hudson Pump Station) has a different flow pattern than the 8th Avenue or Highbury Interceptor. The influence of the local pump station(s) and the different lengths within the existing collection system all have some impact.

2.3 KEY MANHOLE MONITORING PROGRAMS

2.3.2 1998 Program Results - Overview

In 1997 a BOD Task Force recommended the development of a Key Manhole Monitoring (KMM) program in the Vancouver Sewerage Area (VSA). The VSA includes not only the City of Vancouver but also a portion of the City of Burnaby and Richmond. The objective of this program was to:

- Refine BOD data collected in the 1996 BOD sampling program.
- Refine loading estimates from various sectors (residential, commercial, industrial and institutional).
- Develop source loading data for contaminants of interest.

The focus of the 1997 KMM programs was to segment the Vancouver Sewerage Area and characterized 5 tributaries. The tributaries were:

i) Harbour Pump Station
ii) Cheyenne
iii) Yukon Gate
iv) Jervis Street Pump Station, and
v) Dunbar

This program was based on two key features / characteristics: flow proportional composite sampling and sampling only during dry-weather flow. A dry-weather criteria of “less than 2 mm” of precipitation during the sample day was adopted. The goal was to collect 6 days of samples simultaneously at the tributaries and the objective to carry out a mass balance of BOD and TSS loading within the VSA. Site selection was based on meeting monitoring equipment operating criteria, and avoiding confined space entry situations.

The findings of the 1997 KMM program are documented in “Results of the Vancouver Sewerage Area, 1997 Key Manhole Monitoring Program – May 1998”. Conclusions and recommendation of the work identified that the data at Dunbar is suspect due to problems with the flow monitoring equipment. Also, a large portion of BOD and TSS loading in the VSA is from the Harbour tributary. Therefore, in order to refine data, further sampling in 1998 was recommended at Dunbar and at Harbour.

The KMM work in 1998 is similar to the previous years program however, the Harbour and Dunbar programs were operated independently. The two programs were expanded by adding a second site in each of these tributaries. The 1998 report documents the results, conclusions and recommendations based on the data collected from four tributaries, Harbour Pump Station, Harbour West, Dunbar and Kent Pump Station, during July and August of 1998. Tables 2.2 to 2.5 summarize the various results.

TABLE 2.2
HARBOUR PUMP STATION
KEY MANHOLE MONITORING PROGRAM

Parameters	Unit	1997 Range	1998 Average	1998 Range
Flow	ML/d	46.7-89.9	72.9	66.9-76.9
Temp.	C			
pH	-			
DO	mg/L			
TBOD	mg/L	172-476	502*	433-667*
SBOD	mg/L	72-216	-	-
SBOD/TBOD	-			
COD	mg/L	343-903	880	760-1170
COD/TBOD	-	1.75	1.75*	1.75*
TSS	mg/L	110-249	344	229-507
VSS	mg/L	101-230	-	-
NH_3-N	mg/L	11-22	-	-
TKN	mg/L	23-52	-	-
TP	mg/L	3.3-7.5	-	-
* estimated by COD/BOD ratio=1.75				

TABLE 2.3
HARBOUR WEST STATION – KEY MANHOLE MONITORING PROGRAM

Parameters	Unit	1997 Range	1998 Range	1998 Range
Flow	ML/d	-	15.9	14.9-16.6
Temp.	C	-		
pH	-	-		
DO	mg/L	-		
TBOD	mg/L	-	262	210-306
SBOD	mg/L	-	108	100-120
SBOD/TBOD	-	-	0.41	-
COD	mg/L	-	645	540-860
COD/TBOD	-	-	2.46	-
TSS	mg/L	-	317	238-443
VSS	mg/L	-	298	219-400
NH_3-N	mg/L	-	11	10-12
TKN	mg/L	-	33	28-39
TP	mg/L	-	4.4	4.2-4.6

TABLE 2.4
KENT PUMP STATION – KEY MANHOLE MONITORING PROGRAM

Parameters	Unit	1997 Range	1998 Average	1998 Range
Flow	ML/d	-	16.5	12.7-22.5
Temp.	C	-		
pH	-	-		
DO	mg/L	-		
TBOD	mg/L	-	255	188-316
SBOD	mg/L	-	149	102-208
SBOD/TBOD	-	-	0.58	
COD	mg/L	-	491	384-610
COD/TBOD	-	-	1.93	
TSS	mg/L	-	207	173-254
VSS	mg/L	-	186	145-230
NH_3-N	mg/L	-	13	13-14
TKN	mg/L	-	28	25-31
TP	mg/L	-	6.8	4.0-8.1

TABLE 2.5
DUNBAR MANHOLE – KEY MANHOLE MONITORING PROGRAM

Parameters	Unit	1997 Range	1998 Average	1998 Range
Flow	ML/d		50.3	47.7-53.8
Temp.	C			
pH	-			
DO	mg/L			
TBOD	mg/L	112-118	182	170-203
SBOD	mg/L	31-42	41	33-53
SBOD/TBOD	-		0.23	-
COD	mg/L	218-263	413	350-476
COD/TBOD	-		2.27	-
TSS	mg/L	68-125	271	202-331
VSS	mg/L	59-113	241	174-290
NH_3-N	mg/L	11	11	11-12
TKN	mg/L	20-21	27	25-28
TP	mg/L	2.5-3.7	4.5	3.5-5.0

In summary, the Key Manhole Monitoring Program collected data from the following areas:

i) Harbour Pump Station
ii) Harbour West Station
iii) Dunbar
iv) Kent Pump Station

The first two areas (as noted above) are approximately 5 km east and 2.5 km north of the upper end of the 8th Avenue Interceptor sewer. The Highbury interceptor sewer is another 5.5 km further west. If data from these sites were used, we would be relying on samples collected some 13 km away from the area of concern (i.e. the Highbury Interceptor Sewer).

The next two locations include the Dunbar site and the Kent Pump Station. The Dunbar site is well suited and has been selected as one of the proposed 2003 monitoring stations. However, the Kent Pump Station is an estimated 1.5 – 2 km east of the Hudson Pump Station and another 5 – 6 km from the Highbury Interceptor Sewer. Similar to the Harbour Pump Station and the Harbour West area, the Kent Pump Station is a considerable distance away from the Highbury Interceptor.

In addition, the Key Manhole Monitoring Program consisted of “flow proportional composite sampling” during the dry weather time period. In simpler terms, this included the collection of 24 – 30 composite samples over a 24 hour period. These samples are proportionally mixed and tested as one. The goal of this initial program was to collect 6 days of samples simultaneously and carry out a mass balance of BOD and TSS loading within the VSA area. Although this data was very useful for its intended purpose, it has limited value for calibrating a model based on real time control and along the collection system to the Iona WWTP. As such, it is recommended that new data be collected at strategic locations to the Highbury Interceptor.

3 MONITORING PARAMETERS AND LOCATIONS REVIEW

3.1 FLOW MONITORING REVIEW

A total of seven (7) flow monitoring areas were reviewed for this project. This included two (2) stations along the 8th Avenue, Highbury and North Arm Interceptors. The seventh station was the existing influent sampling station located within the Iona WWTP.

3.1.1 Proposed Flow Monitoring Areas

The following provides more specific information on the locations of the various proposed flow monitoring areas:

- 8th Avenue Interceptor

  Area 1: 8th Avenue Interceptor; 8th Avenue and Ash Street, Rim = 16.0 and Inv. = 9.68. This manhole is located in the middle of the west bound lane of 8th Avenue approximately 15 m east of the Ash Street intersection (4-way intersection). This sewer is approximately 6.32 m in depth.

  Area 2: 8th Avenue Interceptor; 8th Avenue and Highbury Street (Manhole at NE corner), Rim = 11.7 and Inv. = 2.66. This manhole is located in the north boulevard east of Highbury Street. At Sta. 0+21.24 (MH1) this sewer is approximately 9.04 m deep. The boulevard appears to provide sufficient room for access.

- Highbury Interceptor

  Area 3: Highbury Interceptor; Highbury Street and 4th Avenue. This existing manhole and boat chamber are located in the boulevard along the west side of Highbury Street, south of 4th Avenue. The depth of this sewer at Sta. 41+76.70 is approximately 5.3 m (manhole configuration and station to be confirmed). The boulevard provides sufficient room for access at this location. This location has been identified as a boat chamber on Drawing No. SF-671, this should provide adequate access for sampling and monitoring.

  Area 4: Highbury Interceptor; Highbury Street, MH7, (150 m south of Marine Drive); Rim = 3.85 and Inv. = -0.14. This Manhole is located along the west edge of pavement on Highbury Street beside a kiosk. At Sta. 0+45.72 (MH7) the sewer is approximately 3.99 m deep. The boulevard appears to provide sufficient room for access along this dead end road.

- North Arm Interceptor

  Area 5: North Arm Interceptor; 75th Avenue and Angus Drive, MH 21, (325 m west of Barnard Street; Rim = 3.84 and Inv. = 1.19. This Manhole is located in the northwest corner of the intersection. At Sta. 0+02.86 (MH21) the sewer is approximately 2.65 m deep. Traffic control would be required for access.

Area 6: North Arm Interceptor; 49th Avenue, MH 3 (260 m west of Dunbar Street); Rim = 1.96 and Inv. = -0.80. This manhole is located at the center of the cul-de-sac. At Sta. 2+66.47 (MH3) the sewer is approximately 2.76 m deep. Limited traffic control would be required on this road.

- Other

  Area 7: This existing flow monitoring station is in place within the Iona WWTP.

Figure 2.1 details the location of the proposed flow monitoring stations.

3.1.2 Field Review of Proposed Flow Monitoring

Stantec and GVRD staff conducted a field review of six of the seven flow monitoring areas on July 29, 2003. A further meeting was held with IIWWTP personnel to review Area 7 on August 11, 2003.

The following is a summary of information obtained from these reviews and following discussions.

- 8th Avenue Interceptor

  Area 1: This manhole does not provide direct access to the 8th Avenue Interceptor. It is located on the overflow sewer to English Bay. Additional manholes between Granville Street and Cambie Street are inaccessible due to depth or obstructions.

  Area 2: This location was found to be adequate for sampling.

- Highbury Interceptor

  Area 3: This location was found to be adequate for sampling.

  Area 4: This location was not reviewed in the field but GVRD staff indicated that it would be adequate for sampling.

- North Arm Interceptor

  Area 5: This location was not reviewed in the field.

  Area 6: This location was recently upgraded with a new manhole access lid and has flow monitoring in place. Sampling would be possible at this location.

- Other

  Area 7: The existing flow monitoring station was found to be acceptable for additional sampling.

3.2 PROPOSED WATER QUALITY PARAMATERS

The following water quality parameters were proposed for the 2003 Monitoring Program.

3.2.1 TBOD, SBOD, TCOD and SCOD

A brief discussion on the use of these following parameters is provided hereafter (1) total biochemical oxygen demand (TBOD), (2) soluble biochemical oxygen demand (SBOD), (3) total chemical oxygen demand (TCOD) and (4) soluble chemical oxygen demand (SCOD) is provided hereafter.

BOD and COD are commonly used to represent the oxygen demand of carbonaceous contaminants in the sewage. The MOUSE TRAP model is capable of accepting either BOD or COD as the input parameter. A fairly constant COD/BOD ratio (e.g. 1.8 to 2.5) is typically observed in the sewage samples. Due to certain disadvantages of the BOD test procedure (i.e. sample seeding, minimum 5-day procedure, equipment requirement, etc.), the COD test is recommended in this monitoring program and modeling exercise. However, several initial tests (5 to 10 samples) of both COD and BOD are necessary to establish the COD/BOD ratios, as required for the MOUSE TRAP model inputs. SBOD and SCOD are analyzed by filtering the sewage samples with a 0.45 µm filter paper.

The analytical procedures are detailed in the Standard Method 5210 and 5220 (APHA et al. 1995) or equivalent methods.

3.2.2 Dissolved Oxygen Concentration (DO)

The DO concentration is critical to the heterotrophic bioreaction, which requires oxygen as the electron acceptor to complete the metabolism. The DO concentrations in sewage are relatively low (e.g. 0 to 3 mg/L), however, reaeration and biological consumption will dynamically affect the DO concentration during the conveyance. The oxygen depleting substances (ODS, previously identified in the IIWWTP effluent toxicity study) or SBOD entering the treatment plant may be affected by the DO availability. The DO concentration is considered the limiting factor in the biochemical process.

The analytical procedures are detailed in the Standard Method 4500-O (APHA et al. 1995) or preferably membrane electrode measurement.

3.2.3 TSS & VSS

The total suspended solids (TSS) and total volatile suspended solids (VSS) are the major parameters used to evaluate the strength of wastewaters. The measurement of suspended solids is considered as significant as BOD and COD, because some organic matter is found in the solid form instead of being found in solution. Typically, VSS concentrations are interpreted as the insoluble proportion of organic substances and the decompositions of VSS through biochemical reactions will convert VSS into soluble phase. TSS and VSS monitoring of the sewer samples can provide variable information in assisting the biochemical reaction assessment, as well as the prediction of solids and organic loads entering the treatment plants. Due to the complexity of transformations between voluble and insoluble (e.g. sediment and biofilm) phases, all solids information as part of quality modeling must be carefully reviewed. Preliminary evaluation of the solids data is recommended prior to implementing them into any water quality modeling exercise.

The analytical procedure of TSS and VSS are detailed in the Standard Method 2540D and 2540E (APHA et al. 1995), or other equivalent methods.

3.2.4 Chlorides (Conservative Substance)

Chlorides are commonly used as tracers in aquatic environment studies. Chlorides are most soluble in solution and conservative in nature in that their mass will not be lost through reactions or partitioning into different phases. Domestic and industrial wastewaters are a prime source of chlorides in the sewer system. In addition, groundwater infiltration and saltwater intrusion may also change chloride concentrations. In monitoring the chloride concentrations along the sewer system, the conditions of wastewater inflow and groundwater infiltration can be determined through a mass balance exercise. The analytical procedures are detailed in Standard Method 4500-cl - (APHA et al 1995).

3.2.5 Ammonia

Ammonia-nitrogen (NH_3-N) can be used as an auxiliary tracer in assessing the inflow and infiltration conditions. The main sources of NH_3-N in sewer system are from the domestic and industrial wastewaters. The ammonia concentrations in sewage samples are also affected by in-situ biochemical reactions (e.g. ammonification, biosynthesis, nitrification and denitrification). The ammonia information can be used in conjunction with other parameters (e.g. BOD, DO) to evaluate the possible biochemical reactions in the system. The analytical procedures are detailed in Standard Method 4500-NH_3 (APHA et al 1995).

3.2.6 pH

The pH condition of typical domestic sewage is about 6.5 to 7.5. Non-domestic discharge may change the pH condition substantially. Although, pH is not a standard input parameter in the MOUSE TRAP model, it can provide information in assisting the sewage characterization (e.g. non-domestic wastewater contribution).

The analytical procedures are detailed in the Standard Method 4500-H^+ (APHA et al. 1995) or preferably electrode measurement using a portable device.

3.2.7 Temperature

The hydrolysis and heterotrophic growth in sewer systems is a temperature dependent biological reaction. Higher temperatures usually result in higher hydrolysis and biomass growth rates.

The analytical procedures are detailed in the Standard Method 2550 (APHA et al. 1995) or preferably electrode measurement using a portable device.

3.2.8 Flow Rate

The flow rate is an essential component required in the hydrodynamic model (e.g. MOUSE PIPE FLOW). The flow rate determines the contaminants' retention time in a sewer, which regulates the degree rate and type of biochemical reactions in the system.

3.3 MONITORING PROGRAM

Monitoring parameters and locations, as described in Section 3.1 and 3.2, as well as proposed sampling frequencies were reviewed with GVRD in order that the proposed works met technical, operational, and overall budget requirements. The final modified program is described in more detail in the following section.

3.3.1 Sampling Locations

Upon review with the GVRD staff, four sampling locations were identified. Field review of the proposed locations as noted in Section 3.1.2 found Area 1 to be inaccessible. In addition, no practical access location for sampling was available along the 8th Avenue Interceptor between Cambie and Highbury Street. Samples were collected at the following four locations:

- **Area 2** = 8th Avenue Interceptor: 8th Avenue / Highbury Street
- **Area 3** = Highbury Interceptor: Highbury Street / 4th Avenue (Boat Chamber)
- **Area 6** = North Arm Interceptor: 49th Avenue, west of Dunbar, MH 3
- **Area 7** = Existing flow monitoring is in place within the Iona WWTP

3.3.2 Water Quality Parameters

Five parameters were identified for review in the samples taken. This included Temperature, pH, Dissolved Oxygen Concentration (DO), Chemical Oxygen Demand (COD), and Chlorides (Chloride). Stantec was responsible for collection of all samples at the four locations and conducted on-site testing for DO, pH, and temperature. Laboratory tests were carried out by B.C. Analytical Technologies Ltd for Chloride and COD.

3.3.3 Sampling Frequency

Samples were collected three times per day to correspond with the AM Low at approximately 6 AM, the Noon Peak at approximately 12 Noon, and the PM Peak at approximately 6 PM. This was done over a 48 hour timeframe during two consecutive week periods. Testing periods were proposed to be conducted mid week to avoid weekend shifts in sewage patterns. The two sampling periods were:

- **Week 1:** Wednesday, August 13, 2003 Noon Peak to Friday, August 15, 2003 AM Low
- **Week 2:** Tuesday, August 19, 2003 PM Peak to Thursday, August 21, 2003 Noon Peak

Table 3.1 outlines the updated sampling schedules, identifying the proposed scheduled times and the actual times samples were taken. The schedule was affected by access, travel, preparation, and clean-up time.

TABLE 3.1
SAMPLE FREQUENCY AND LOCATION

Date	8th Avenue Interceptor		Highbury Interceptor		North Arm Interceptor		Iona WWTP		Comments
	Area 2		Area 3		Area 6		Area 7		
	Actual	Scheduled	Actual	Scheduled	Actual	Scheduled	Actual	Scheduled	
13-Aug-03	11:42	11:45	11:23	11:35	12:17	12:15	13:07	12:45	Noon Peak
13-Aug-03	18:15	19:15	17:53	19:05	18:53	19:45	19:41	20:15	PM Peak
14-Aug-03	6:12	6:00	5:49	5:50	6:42	6:30	7:20	7:00	AM Low
14-Aug-03	11:40	11:45	11:22	11:35	12:20	12:15	13:03	12:45	Noon Peak
14-Aug-03	18:12	19:15	17:50	19:05	18:45	19:45	19:32	20:15	PM Peak
15-Aug-03	6:13	6:00	5:50	5:50	6:47	6:30	7:39	7:00	AM Low
19-Aug-03	18:09	19:15	17:50	19:05	18:36	19:45	19:35	20:15	PM Peak
20-Aug-03	6:11	6:00	5:51	5:50	6:44	6:30	7:28	7:00	AM Low
20-Aug-03	11:35	11:45	11:15	11:35	12:11	12:15	13:05	12:45	Noon Peak
20-Aug-03	18:08	19:15	17:49	19:05	18:48	19:45	19:39	20:15	PM Peak
21-Aug-03	6:18	6:00	5:54	5:50	6:50	6:30	7:38	7:00	AM Low
21-Aug-03	11:41	11:45	11:23	11:35	12:17	12:15	13:08	12:45	Noon Peak

Area 2 = 8th Avenue Interceptor: 8th Avenue / Highbury Street, MH 1
Area 3 =Highbury Interceptor: Highbury Street / 4th Avenue (Boat Chamber)
Area 6 = North Arm Interceptor: 49th Avenue, west of Dunbar, MH 3
Area 7 = Iona WWTP: Influent Sample Station at Iona WWTP

3.4 SAMPLE TESTING PROCEDURES AND EQUIPMENT

Samples were drawn from the sewer at Areas 2, 3 and 6 through manhole accesses using a rubber bucket lowered into the flow. From this bucket readings were taken and sample jars filled. At Area 7 sample jars where filled directly from the influent sampling station and readings taken from these samples. Temperature and pH were measured in the field using a Hach EC20 Portable pH/ISE Meter Model 50075. Measuring pH this meter provides an accuracy of ± 0.005 while measuring temperature the accuracy is ± 1.0 °C. DO was also measured in the field by using an YSI Model 54 Dissolved Oxygen Meter. The accuracy of the DO meter is ± 0.1 mg/L, though it is understood that accuracy degrades when the DO content is below 1.0 mg/L. Samples were taken to B.C. Analytical Technologies Ltd. for analysis of COD and Chloride. 500 ml samples were taken and stored on ice for both COD and Chloride; with COD samples preserved using Sulphuric Acid to insure a pH of below 3.0. Chemical Oxygen demand was measured using a Closed Reflux/Colorimetric APHA5220 D with a reported detection limit of 20 mg/L while chloride was measured using a Filtration/Incubation/DO APHA5210 B 4500-ogc with a reported detection limit of 2.5 mg/L. Analytical results are summarized in the attached Appendix A. A summary of sampling results by location is provided in Section 4.

4 RESULTS OF MONITORING PROGRAM

4.1 MONITORING PROGRAM RESULTS

The following figures and tables summarize the monitoring results obtained during the sampling period. The attached figures provide an overview of the results by area. Tables have been provided which review the results by area and sampling period, including minimum, maximum and average values.

FIGURE 4.1
SAMPLING RESULTS AREA 2

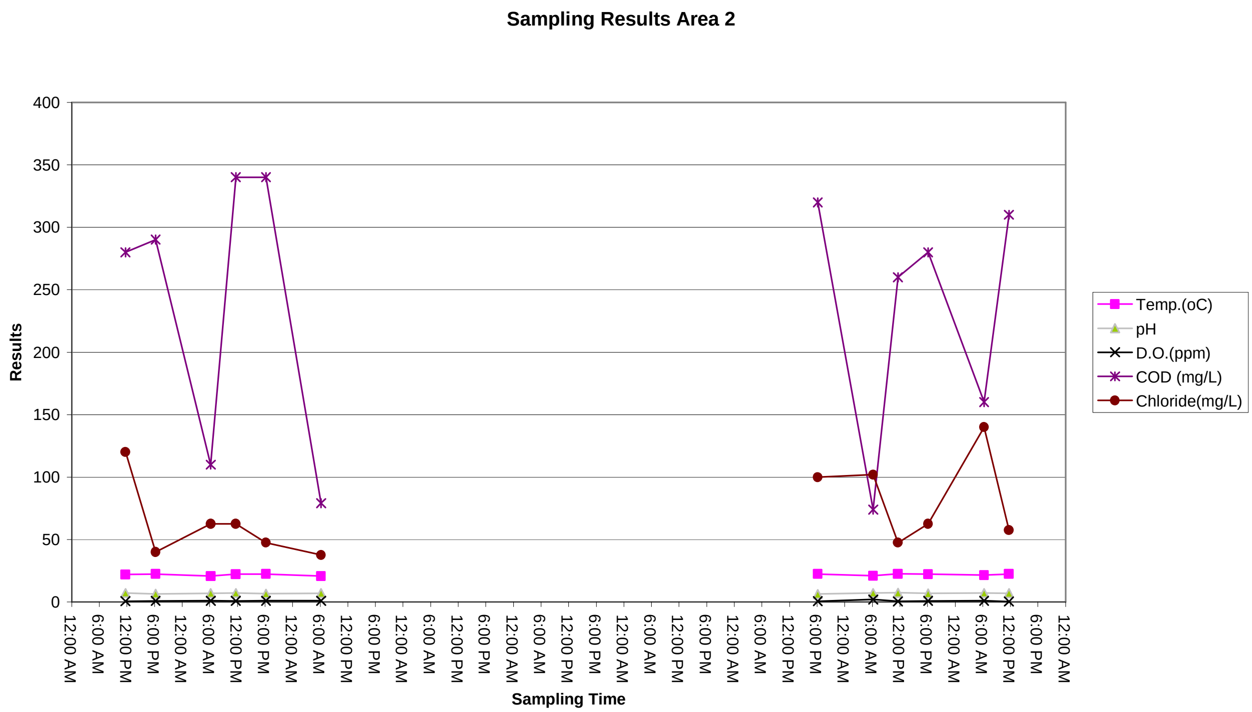


FIGURE 4.2
SAMPLING RESULTS AREA 3

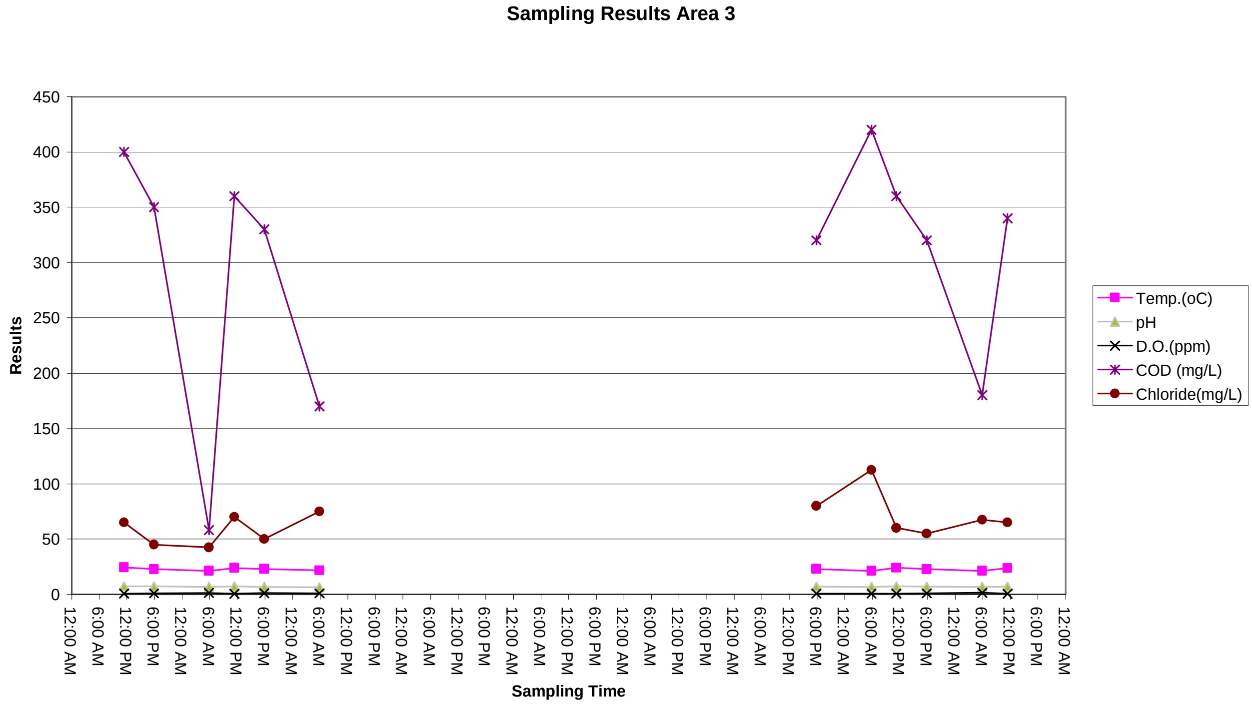


FIGURE 4.3
SAMPLING RESULTS AREA 6

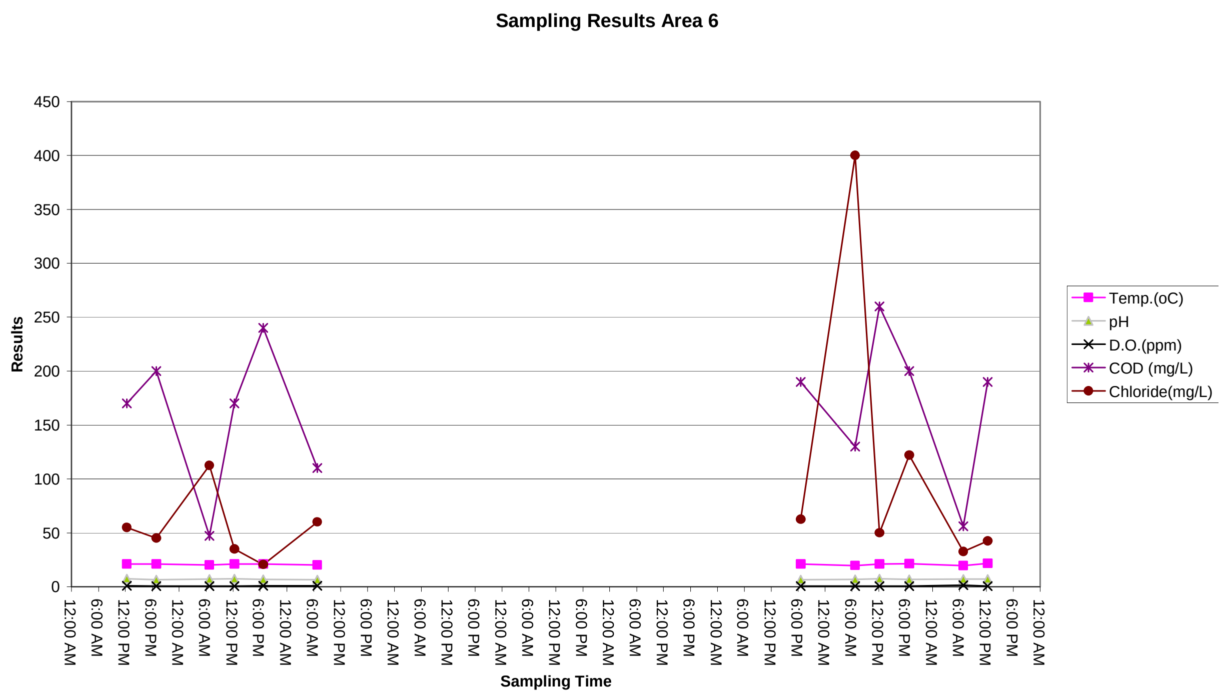
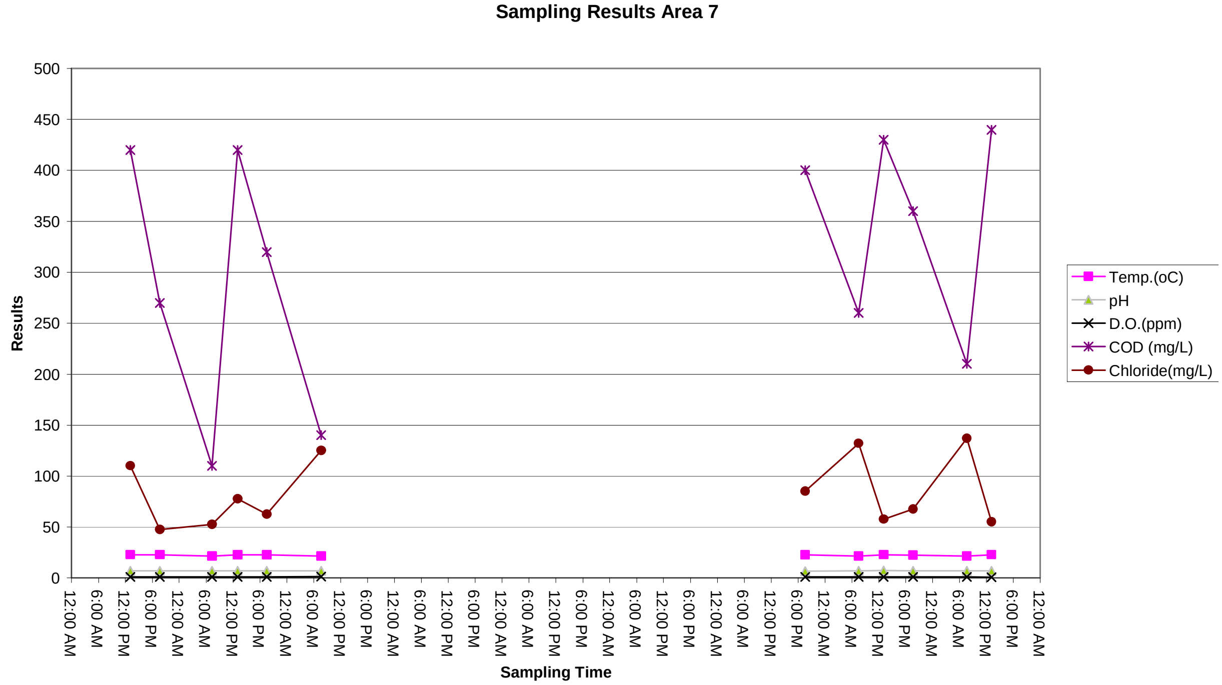


FIGURE 4.4
SAMPLING RESULTS AREA 7



The following Table 4.1, 4.2A and 4.2B provide a more detailed overview of the monitoring program results.

TABLE 4.1
SAMPLING RESULTS

Date	8th Avenue Interceptor: 8th Avenue / Highbury Street, MH 1								Comments
	Area 2								
	Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride	
13-Aug-03	11:42	11:45	2-1	22.1	7.13	0.6	280	120	NoonPeak
13-Aug-03	18:15	19:15	2-2	22.4	6.59	0.8	290	40	PM Peak
14-Aug-03	6:12	6:00	2-3	20.7	7.09	0.9	110	62.5	AM Low
14-Aug-03	11:40	11:45	2-4	22.2	7.12	0.8	340	62.5	Noon Peak
14-Aug-03	18:12	19:15	2-5	22.3	6.72	0.9	340	47.5	PM Peak
15-Aug-03	6:13	6:00	2-6	20.7	7.01	1.0	79	37.5	AM Low
19-Aug-03	18:09	19:15	2-7	22.4	6.41	0.6	320	100.0	PM Peak
20-Aug-03	6:11	6:00	2-8	20.9	7.18	2.0	74	102	AM Low
20-Aug-03	11:35	11:45	2-9	22.5	7.42	0.5	260	47.5	Noon Peak
20-Aug-03	18:08	19:15	2-10	22.2	6.90	0.8	280	62.5	PM Peak
21-Aug-03	6:18	6:00	2-11	21.5	7.14	1.0	160	140	AM Low
21-Aug-03	11:41	11:45	2-12	22.4	7.05	0.4	310	57.5	Noon Peak

Date	Highbury Interceptor: Highbury Street / 4th Avenue (Boat Chamber)								Comments
	Area 3								
	Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride	
13-Aug-03	11:23	11:35	3-1	24.4	7.27	0.8	400	65	NoonPeak
13-Aug-03	17:53	19:05	3-2	22.7	7.19	0.9	350	45	PM Peak
14-Aug-03	5:49	5:50	3-3	21.4	6.86	1.2	58	42.5	AM Low
14-Aug-03	11:22	11:35	3-4	23.9	7.25	0.5	360	70	Noon Peak
14-Aug-03	17:50	19:05	3-5	23.0	6.88	1.1	330	50	PM Peak
15-Aug-03	5:50	5:50	3-6	21.6	6.52	1.0	170	75	AM Low
19-Aug-03	17:50	19:05	3-7	23.0	6.97	0.8	320	80.0	PM Peak
20-Aug-03	5:51	5:50	3-8	21.3	6.75	0.8	420	112.5	AM Low
20-Aug-03	11:15	11:35	3-9	24.0	7.21	0.7	360	60.0	Noon Peak
20-Aug-03	17:49	19:05	3-10	22.7	6.90	0.9	320	55.0	PM Peak
21-Aug-03	5:54	5:50	3-11	21.3	6.72	1.5	180	67.5	AM Low
21-Aug-03	11:23	11:35	3-12	23.7	6.98	0.5	340	65.0	Noon Peak

Date	North Arm Interceptor: 49th Avenue, west of Dunbar, MH 3								Comments
	Area 6								
	Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride	
13-Aug-03	12:17	12:15	6-1	21.2	7.31	0.8	170	55	NoonPeak
13-Aug-03	18:53	19:45	6-2	21.2	6.36	0.6	200	45	PM Peak
14-Aug-03	6:42	6:30	6-3	20.0	7.01	0.6	47	112.5	AM Low
14-Aug-03	12:20	12:15	6-4	21.1	7.26	0.5	170	35	Noon Peak
14-Aug-03	18:45	19:45	6-5	21.2	6.82	1.0	240	20.5	PM Peak
15-Aug-03	6:47	6:30	6-6	20.2	6.64	0.9	110	60	AM Low
19-Aug-03	18:36	19:45	6-7	21.0	6.66	0.5	190	62.5	PM Peak
20-Aug-03	6:44	6:30	6-8	19.7	6.76	0.6	130	400	AM Low
20-Aug-03	12:11	12:15	6-9	21.1	7.38	0.6	260	50.0	Noon Peak
20-Aug-03	18:48	19:45	6-10	21.3	6.74	0.5	200	122	PM Peak
21-Aug-03	6:50	6:30	6-11	19.5	6.90	1.4	56	32.5	AM Low
21-Aug-03	12:17	12:15	6-12	21.6	7.19	0.5	190	42.5	Noon Peak

Date	Iona WWTP: Influent Sample Station at Iona WWTP								Comments
	Area 7								
	Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride	
13-Aug-03	13:07	12:45	7-1	22.7	7.06	0.8	420	110	NoonPeak
13-Aug-03	19:41	20:15	7-2	22.8	6.93	0.8	270	47.5	PM Peak
14-Aug-03	7:20	7:00	7-3	21.4	6.93	0.9	110	52.5	AM Low
14-Aug-03	13:03	12:45	7-4	22.6	6.91	0.7	420	77.5	Noon Peak
14-Aug-03	19:32	20:15	7.5	22.5	6.71	0.9	320	62.5	PM Peak
15-Aug-03	7:39	7:00	7-6	21.2	6.81	1.1	140	125	AM Low
19-Aug-03	19:35	20:15	7-7	22.6	6.61	1.0	400	85.0	PM Peak
20-Aug-03	7:28	7:00	7-8	21.4	7.09	0.9	260	132	AM Low
20-Aug-03	13:05	12:45	7-9	22.7	7.14	0.7	430	57.5	Noon Peak
20-Aug-03	19:39	20:15	7-10	22.4	6.91	0.9	360	67.5	PM Peak
21-Aug-03	7:38	7:00	7-11	21.4	7.05	1.0	210	137	AM Low
21-Aug-03	13:08	12:45	7-12	22.7	7.07	0.5	440	55.0	Noon Peak

TABLE 4.2A
SAMPLING RESULTS

	Date	8th Avenue Interceptor: 8th Avenue / Highbury Street, MH 1 Area 2								Comments
		Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride (mg/L)	
	14-Aug-03	6:12	6:00	2-3	20.7	7.09	0.9	110	62.5	AM Low
	15-Aug-03	6:13	6:00	2-6	20.7	7.01	1.0	79	37.5	AM Low
	20-Aug-03	6:11	6:00	2-8	20.9	7.18	2.0	74	102	AM Low
	21-Aug-03	6:18	6:00	2-11	21.5	7.14	1.0	160	140	AM Low
Minimum		6:11			20.7	7.01	0.90	74	37.5	
Maximum		6:18			21.5	7.18	2.00	160	140.0	
Average		6:13			21.0	7.11	1.23	106	85.5	
	13-Aug-03	11:42	11:45	2-1	22.1	7.13	0.6	280	120	Noon Peak
	14-Aug-03	11:40	11:45	2-4	22.2	7.12	0.8	340	62.5	Noon Peak
	20-Aug-03	11:35	11:45	2-9	22.5	7.42	0.5	260	47.5	Noon Peak
	21-Aug-03	11:41	11:45	2-12	22.4	7.05	0.4	310	57.5	Noon Peak
Minimum		11:35			22.1	7.05	0.40	260	47.5	
Maximum		11:42			22.5	7.42	0.80	340	120.0	
Average		11:39			22.3	7.18	0.58	298	71.9	
	13-Aug-03	18:15	19:15	2-2	22.4	6.59	0.8	290	40	PM Peak
	14-Aug-03	18:12	19:15	2-5	22.3	6.72	0.9	340	47.5	PM Peak
	19-Aug-03	18:09	19:15	2-7	22.4	6.41	0.6	320	100	PM Peak
	20-Aug-03	18:08	19:15	2-10	22.2	6.9	0.8	280	62.5	PM Peak
Minimum		18:08			22.2	6.41	0.60	280	40.0	
Maximum		18:15			22.4	6.90	0.90	340	100.0	
Average		18:11			22.3	6.66	0.78	308	62.5	

	Date	Highbury Interceptor: Highbury Street / 4th Avenue (Boat Chamber) Area 3								Comments
		Actual	Scheduled	Sample ID	Temp.(^oC)	pH	D.O.(mg/L)	COD (mg/L)	Chloride (mg/L)	
	14-Aug-03	5:49	5:50	3-3	21.4	6.86	1.2	58	42.5	AM Low
	15-Aug-03	5:50	5:50	3-6	21.6	6.52	1.0	170	75	AM Low
	20-Aug-03	5:51	5:50	3-8	21.3	6.75	0.8	420	112.5	AM Low
	21-Aug-03	5:54	5:50	3-11	21.3	6.72	1.5	180	67.5	AM Low
Minimum		5:49			21.3	6.52	0.80	58	42.5	
Maximum		5:54			21.6	6.86	1.50	180	112.5	
Average		5:51			21.4	6.71	1.13	136	74.4	
	13-Aug-03	11:23	11:35	3-1	24.4	7.27	0.8	400	65	Noon Peak
	14-Aug-03	11:22	11:35	3-4	23.9	7.25	0.5	360	70	Noon Peak
	20-Aug-03	11:15	11:35	3-9	24	7.21	0.7	360	60	Noon Peak
	21-Aug-03	11:23	11:35	3-12	23.7	6.98	0.5	340	65	Noon Peak
Minimum		11:15			23.7	6.98	0.50	340	60.0	
Maximum		11:23			24.4	7.27	0.80	400	70.0	
Average		11:20			24.0	7.18	0.63	365	65.0	
	13-Aug-03	17:53	19:05	3-2	22.7	7.19	0.9	350	45	PM Peak
	14-Aug-03	17:50	19:05	3-5	23	6.88	1.1	330	50	PM Peak
	19-Aug-03	17:50	19:05	3-7	23	6.97	0.8	320	80	PM Peak
	20-Aug-03	17:49	19:05	3-10	22.7	6.9	0.9	320	55	PM Peak
Minimum		17:49			22.7	6.88	0.80	320	45.0	
Maximum		17:53			23.0	7.19	1.10	350	80.0	
Average		17:50			22.9	6.99	0.93	330	57.5	

Note:
= discarded value not used in average calculation

TABLE 4.2B
SAMPLING RESULTS

	Date	North Arm Interceptor: 49th Avenue, west of Dunbar, MH 3								Comments
		Area 6								
		Actual	Scheduled	Sample ID	Temp.(°C)	pH	D.O.(mg/L)	COD (mg/L)	Chloride (mg/L)	
	14-Aug-03	6:42	6:30	6-3	20	7.01	0.6	47	112.5	AM Low
	15-Aug-03	6:47	6:30	6-6	20.2	6.64	0.9	110	60	AM Low
	20-Aug-03	6:44	6:30	6-8	19.7	6.76	0.6	130	400	AM Low
	21-Aug-03	6:50	6:30	6-11	19.5	6.9	1.4	56	32.5	AM Low
Minimum		6:42			19.5	6.64	0.60	47	32.5	
Maximum		6:50			20.2	7.01	1.40	130	112.5	
Average		6:45			19.9	6.83	0.88	86	68.3	
	13-Aug-03	12:17	12:15	6-1	21.2	7.31	0.8	170	55	Noon Peak
	14-Aug-03	12:20	12:15	6-4	21.1	7.26	0.5	170	35	Noon Peak
	20-Aug-03	12:11	12:15	6-9	21.1	7.38	0.6	260	50	Noon Peak
	21-Aug-03	12:17	12:15	6-12	21.6	7.19	0.5	190	42.5	Noon Peak
Minimum		12:11			21.1	7.19	0.50	170	35.0	
Maximum		12:20			21.6	7.38	0.80	260	55.0	
Average		12:16			21.3	7.29	0.60	198	45.6	
	13-Aug-03	18:53	19:45	6-2	21.2	6.36	0.6	200	45	PM Peak
	14-Aug-03	18:45	19:45	6-5	21.2	6.82	1	240	20.5	PM Peak
	19-Aug-03	18:36	19:45	6-7	21	6.66	0.5	190	62.5	PM Peak
	20-Aug-03	18:48	19:45	6-10	21.3	6.74	0.5	200	122	PM Peak
Minimum		18:36			21.0	6.36	0.50	190	20.5	
Maximum		18:53			21.3	6.82	1.00	240	122.0	
Average		18:45			21.2	6.65	0.65	208	62.5	
	Date	Iona WWTP: Influent Sample Station at Iona WWTP								Comments
		Area 7								
		Actual	Scheduled	Sample ID	Temp.(°C)	pH	D.O.(mg/L)	COD (mg/L)	Chloride (mg/L)	
	14-Aug-03	7:20	7:00	7-3	21.4	6.93	0.9	110	52.5	AM Low
	15-Aug-03	7:39	7:00	7-6	21.2	6.81	1.1	140	125	AM Low
	20-Aug-03	7:28	7:00	7-8	21.4	7.09	0.9	260	132	AM Low
	21-Aug-03	7:38	7:00	7-11	21.4	7.05	1.0	210	137	AM Low
Minimum		7:20			21.2	6.81	0.90	110	52.5	
Maximum		7:39			21.4	7.09	1.10	260	137.0	
Average		7:31			21.4	6.97	0.98	180	111.6	
	13-Aug-03	13:07	12:45	7-1	22.7	7.06	0.8	420	110	Noon Peak
	14-Aug-03	13:03	12:45	7-4	22.6	6.91	0.7	420	77.5	Noon Peak
	20-Aug-03	13:05	12:45	7-9	22.7	7.14	0.7	430	57.5	Noon Peak
	21-Aug-03	13:08	12:45	7-12	22.7	7.07	0.5	440	55	Noon Peak
Minimum		13:03			22.6	6.91	0.50	420	55.0	
Maximum		13:08			22.7	7.14	0.80	440	110.0	
Average		13:05			22.7	7.05	0.68	428	75.0	
	13-Aug-03	19:41	20:15	7-2	22.8	6.93	0.8	270	47.5	PM Peak
	14-Aug-03	19:32	20:15	7.5	22.5	6.71	0.9	320	62.5	PM Peak
	19-Aug-03	19:35	20:15	7-7	22.6	6.61	1	400	85	PM Peak
	20-Aug-03	19:39	20:15	7-10	22.4	6.91	0.9	360	67.5	PM Peak
Minimum		19:32			22.4	6.61	0.80	270	47.5	
Maximum		19:41			22.8	6.93	1.00	400	85.0	
Average		19:36			22.6	6.79	0.90	338	65.6	

Note:
= discarded value not used in average calculation

4.2 CHEMICAL OXYGEN DEMAND

Grab samples were taken and preserved by Stantec for analysis of the Chemical Oxygen Demand (COD) by B.C. Analytical Technologies Ltd. Laboratory. Results of this analysis are summarized below.

TABLE 4.3
SUMMARY OF CHEMICAL OXYGEN DEMAND RESULTS

Location	Flow Period	Minimum COD (mg/L)	Maximum COD (mg/L)	Range (mg/L)	Average COD (mg/L)
Area 2 **8th Avenue Interceptor**	AM Low	74	160	86	106
	Noon Peak	260	340	80	298
	PM Peak	280	340	60	308
	Average 24 hour range for all data				202
Area 3 **Highbury Trunk Sewer**	AM Low	58	180	122	136
	Noon Peak	340	400	60	365
	PM Peak	320	350	30	330
	Average 24 hour range for all data				194
Area 6 **North Arm Interceptor**	AM Low	47	130	123	86
	Noon Peak	170	260	90	198
	PM Peak	190	240	50	208
	Average 24 hour range for all data				122
Area 7 **Iona Island WWTP**	AM Low	110	260	150	180
	Noon Peak	420	440	20	428
	PM Peak	270	400	130	338
	Average 24 hour range				248

Note:
1) One data point discarded in average calculations (420 mg/L – August 20 ,2003) as value significantly greater than other AM Low values.
2) Detection limit of 20 mg/L

i) The results obtained show that COD increased from the AM Low to the Noon Peak period and generally remained constant during the peak periods.

ii) Generally this pattern corresponds with flow, COD being directly proportional to flow. (i.e. COD increases as the flow increases from the morning low flow to the noon peak and afternoon peak flows).

iii) The AM Low period consistently had the lowest average COD (86 – 180 mg/L).

iv) The Noon Peak period consistently had the highest average COD (198 to 428 mg/L).

v) The average PM Peak COD levels were generally similar to the Noon Peak period, but higher than the AM Low period results. (208 – 338 mg/L).

vi) Area 7 (Iona Island WWTP) experienced a considerably higher average COD during the Noon Peak period when compared to the PM Peak period. This may be due to the presence of trucked liquid waste and recycled streams of effluent introduced at the IIWWTP, which results in an increased oxygen demand.

Recycled streams at the IIWWTP include thickener return, screenings return and lagoon return.

vii) A COD mass balance during the sampling periods was completed and is summarized in Tables 4.4, 4.5 and 4.6.

viii) The COD is higher in the sewage in the Highbury Interceptor at 4th Street than in the sewage in the 8th Avenue Interceptor just before it enters the Highbury Interceptor. This indicates higher loading from areas feeding into Highbury at 4th Avenue.

TABLE 4.4
AM LOW COD MASS BALANCE

	Date	Flow Period	Location	Flow (MLD)	COD (mg/L)	COD Mass (kg/hr)
	14-Aug-03	AM Low	Area 2 - 8th Avenue Interceptor	173	110.0	792
			Area 3 - Highbury Interceptor	81	58.0	196
			Area 6 - North Arm Interceptor	46	47.0	90
AREA 2, 3, AND 6 SUBTOTAL				**300**		**1078**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	340	110.0	1556
PERCENT DIFFERENCE				**13%**		**44%**
	15-Aug-03	AM Low	Area 2 - 8th Avenue Interceptor	176	79.0	580
			Area 3 - Highbury Interceptor	81	170.0	575
			Area 6 - North Arm Interceptor	47	110.0	214
AREA 2, 3, AND 6 SUBTOTAL				**304**		**1369**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	358	140.0	2087
PERCENT DIFFERENCE				**18%**		**52%**
	20-Aug-03	AM Low	Area 2 - 8th Avenue Interceptor	174	74.0	
			Area 3 - Highbury Interceptor	81	420.0	
			Area 6 - North Arm Interceptor	47	130.0	
AREA 2, 3, AND 6 SUBTOTAL						
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	354	260.0	
PERCENT DIFFERENCE						
	21-Aug-03	AM Low	Area 2 - 8th Avenue Interceptor	178	160.0	1187
			Area 3 - Highbury Interceptor	82	180.0	616
			Area 6 - North Arm Interceptor	48	56.0	113
AREA 2, 3, AND 6 SUBTOTAL				**308**		**1915**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	358	210.0	3130
PERCENT DIFFERENCE				**16%**		**63%**
AREA 2, 3, AND 6 SUBTOTAL AVERAGE		AM Low		**304**		**1454**
AREA 7 SUBTOTAL AVERAGE				**352**		**2258**
AVERAGE PERCENT DIFFERENCE				**16%**		**55%**

Note:
August 20, 2003 AM Low period not calculated due to discarded COD value for Area 3

TABLE 4.5
NOON PEAK COD MASS BALANCE

	Date	Flow Period	Location	Flow (MLD)	COD (mg/L)	COD Mass (kg/hr)
	13-Aug-03	Noon Peak	Area 2 - 8th Avenue Interceptor	295	280.0	3437
			Area 3 - Highbury Interceptor	139	400.0	2318
			Area 6 - North Arm Interceptor	89	170.0	632
AREA 2, 3, AND 6 SUBTOTAL				**523**		**6388**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	521	420.0	9117
PERCENT DIFFERENCE				**0%**		**43%**
	14-Aug-03	Noon Peak	Area 2 - 8th Avenue Interceptor	295	340.0	4174
			Area 3 - Highbury Interceptor	139	360.0	2087
			Area 6 - North Arm Interceptor	89	170.0	630
AREA 2, 3, AND 6 SUBTOTAL				**523**		**6891**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	521	420.0	9117
PERCENT DIFFERENCE				**0%**		**32%**
	20-Aug-03	Noon Peak	Area 2 - 8th Avenue Interceptor	295	260.0	3201
			Area 3 - Highbury Interceptor	138	360.0	2074
			Area 6 - North Arm Interceptor	89	260.0	964
AREA 2, 3, AND 6 SUBTOTAL				**523**		**6239**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	521	430.0	9334
PERCENT DIFFERENCE				**0%**		**50%**
	21-Aug-03	Noon Peak	Area 2 - 8th Avenue Interceptor	295	310.0	3806
			Area 3 - Highbury Interceptor	139	340.0	1971
			Area 6 - North Arm Interceptor	89	190.0	705
AREA 2, 3, AND 6 SUBTOTAL				**523**		**6481**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	521	440.0	9552
PERCENT DIFFERENCE				**0%**		**47%**
AREA 2, 3, AND 6 SUBTOTAL AVERAGE		Noon Peak		**523**		**6500**
AREA 7 SUBTOTAL AVERAGE				**521**		**9280**
AVERAGE PERCENT DIFFERENCE				**0%**		**43%**

TABLE 4.6
PM PEAK COD MASS BALANCE

	Date	Flow Period	Location	Flow (MLD)	COD (mg/L)	COD Mass (kg/hr)
	13-Aug-03	PM Peak	Area 2 - 8th Avenue Interceptor	285	290.0	3445
			Area 3 - Highbury Interceptor	131	350.0	1915
			Area 6 - North Arm Interceptor	84	200.0	698
AREA 2, 3, AND 6 SUBTOTAL				**500**		**6059**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	501	270.0	5638
PERCENT DIFFERENCE				**0%**		**-7%**
	14-Aug-03	PM Peak	Area 2 - 8th Avenue Interceptor	285	340.0	4039
			Area 3 - Highbury Interceptor	131	330.0	1806
			Area 6 - North Arm Interceptor	84	240.0	838
AREA 2, 3, AND 6 SUBTOTAL				**500**		**6683**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	504	320.0	6716
PERCENT DIFFERENCE				**1%**		**0%**
	19-Aug-03	PM Peak	Area 2 - 8th Avenue Interceptor	285	320.0	3802
			Area 3 - Highbury Interceptor	131	320.0	1751
			Area 6 - North Arm Interceptor	84	190.0	663
AREA 2, 3, AND 6 SUBTOTAL				**500**		**6216**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	502	400.0	8366
PERCENT DIFFERENCE				**0%**		**35%**
	20-Aug-03	PM Peak	Area 2 - 8th Avenue Interceptor	285	280.0	3326
			Area 3 - Highbury Interceptor	131	320.0	1751
			Area 6 - North Arm Interceptor	83	200.0	691
AREA 2, 3, AND 6 SUBTOTAL				**499**		**5769**
AREA 7 SUBTOTAL			Area 7 - Iona WWTP	501	360.0	7517
PERCENT DIFFERENCE				**0%**		**30%**
AREA 2, 3, AND 6 SUBTOTAL AVERAGE		PM Peak		**500**		**6182**
AREA 7 SUBTOTAL AVERAGE				**502**		**7059**
AVERAGE PERCENT DIFFERENCE				**0%**		**14%**

i) Flow information for the mass balance was obtained from the GVRD's Mouse Model of this system for the times samples were taken. For the AM Low period the average combined flow at the various sampling locations was typically 13-18% lower than the modeled flow at the Iona Island WWTP. This may be attributed to the time sequence selected for sampling. (i.e. a later test at Iona would correspond with a higher flows, as flows rise in the morning) No significant variance was noted in the Noon Peak or PM Peak flows.

ii) A COD mass balance for the AM Low and Noon Peak periods indicated a 43-55% increase in COD loading at the Iona Island WWTP. When the AM Low results were adjusted to account for the 13-18% flow variance noted, it was determined that the COD mass balance was approximately 34-43% higher than that measured in upstream tributary pipes for the sampling periods. This variance is thought to be a direct result of the trucked liquid waste and recycled streams of effluent, which are introduced at the Iona Island WWTP during the AM and Noon periods.

iii) The plant internal recycled streams (e.g. lagoon return, screening return, thickener supernatant) are not monitored in this work. The flow and load of TLW and internal recycle streams are best approximated using the TLW monitoring data in Appendix #1 and some discrete historical plant data. The combination of TLW and internal recycled streams could contribute about 10 to 30% of total

COD load, subject to the time of the day and waste strength (i.e. the schedules of internal recycles and TLW discharge). This contribution would be a significant factor during 6:00 AM to noon since the COD load from VSA was low. If an average load increase of 20% at the Iona WWTP is subtracted from the calculations for the COD mass balance during the AM Low and Noon Peak period the difference in values is reduced to 12-19%.

iv) The COD mass balance for the PM Peak period indicated a 14% variance for all sampling completed. It should be noted that the variance for the August 13-14 time period was 0-7%, while the variance for the August 19/-20 period was 30-35%. This may suggest that there was something unique to the August 19/20 time period, which resulted in a considerably higher COD loading at the IWWTP.

v) The 1998 Key Manhole Monitoring program results were based on composite sampling techniques, which entailed 24-30 discrete samples over a 24 hour period. The sampling program completed as part of this program entails 4 samples over a 48 hour time period. As such the results are not directly comparable.

vi) The range of COD was constant with the exception of the results obtained from Area 3 for the AM Low period. The reading of 420 mg/L on August 20, 2003 at this location was considerable higher than the average for this period. Varied results can be expected due to the nature of grab samples. This reading was discarded in overall calculations because of the large difference compared to the average. If this result was included the Average COD for Area 3 during the AM Low period would be 207 mg/L, as opposed to 136 mg/L.

vii) As no unexpected COD loads were noted in the results this would support the hypothesis that although there are microorganisms at work in the sewers, these are not significantly reducing the organic loading.

4.3 DISSOLVED OXYGEN

Dissolved Oxygen Concentration (DO) was measured using the YSI Model 54 Dissolved Oxygen Meter. Readings were taken as effluent was drawn from the sampling location. Average DO readings at the sampling locations are summarized below.

TABLE 4.7
SUMMARY OF DISSOLVED OXYGEN RESULTS

Location	Flow Period	Minimum DO (mg/L)	Maximum DO (mg/L)	Range (mg/L)	Average DO (mg/L)
Area 2 **8th Avenue Interceptor**	AM Low	0.9	2.0	1.1	1.2
	Noon Peak	0.4	0.8	0.4	0.6
	PM Peak	0.6	0.9	0.3	0.8
	Average 24 hour range				0.6
Area 3 **Highbury Trunk Sewer**	AM Low	0.8	1.5	0.7	1.1
	Noon Peak	0.5	0.8	0.3	0.6
	PM Peak	0.8	1.1	0.3	0.9
	Average 24 hour range				0.5
Area 6 **North Arm Interceptor**	AM Low	0.6	1.4	0.8	0.9
	Noon Peak	0.5	0.8	0.3	0.6
	PM Peak	0.5	1.0	0.5	0.7
	Average 24 hour range				0.3
Area 7 **Iona Island WWTP**	AM Low	0.9	1.1	0.2	1.0
	Noon Peak	0.5	0.8	0.3	0.7
	PM Peak	0.8	1.0	0.2	0.9
	Average 24 hour range				0.3

Note:
1) Measurement accuracy ± 0.1 mg/L.

i) Measured DO levels varied from 0.5 to 2.0 mg/L. Typical DO levels were 1.0 mg/L or less.

ii) The results indicated that the DO level decreased from the AM Low period to the Noon Peak period and then began to increase towards the PM Peak period.

iii) The AM Low period consistently had the highest average DO level (0.8 – 1.2 mg/L).

iv) The Noon Peak period consistently had the lowest average DO level (0.6 - 0.7 mg/L).

v) The average PM Peak period DO levels were consistently higher then the Noon Peak period and lower than the AM Low period results (0.7 – 0.9 mg/L).

vi) The DO level pattern corresponds with the flow, having DO level inversely proportional to flow. (i.e. the DO level decreased when flow increased). This may be the results of different proportions of sanitary input versus infiltration at different times of the day because there would be a higher proportion of clean infiltration flow at night than during the day.

vii) The range of the DO levels observed is consistent through Areas 2, 3 and 6 with a slightly tighter range found at Area 7 (IIWWTP).

viii) The results confirmed that the DO levels of the raw wastewater in the sewers upstream of the Iona Island WWTP (0.2 to 2.0 mg/L) were typical for a large metropolitan sewage collection system and very much in agreement with the data from the sewer system feeding the Annacis Island WWTP.

4.4 CHLORIDES

Analysis for Chloride concentration was also undertaken by B.C. Analytical Technologies Ltd. Laboratory results of the samples taken is summarized below (Table 4.8).

TABLE 4.8
SUMMARY OF CHLORIDE CONTENT RESULTS

Location	Flow Period	Minimum Chloride (mg/L)	Maximum Chloride (mg/L)	Range (mg/L)	Average Chloride (mg/L)
Area 2 **8th Avenue Interceptor**	AM Low	37.5	140.0	102.5	85.5
	Noon Peak	47.5	120.0	72.5	71.9
	PM Peak	40.0	100.0	60.0	62.5
	Average 24 hour range				23.0
Area 3 **Highbury Trunk Sewer**	AM Low	42.5	112.5	70.0	74.4
	Noon Peak	60.0	70.0	10.0	65.0
	PM Peak	45.0	80.0	35.0	57.5
	Average 24 hour range				16.9
Area 6 **North Arm Interceptor**	AM Low1	32.5	112.5	80.0	68.3
	Noon Peak	35.0	55.0	20.0	45.6
	PM Peak	20.5	122.0	101.5	62.5
	Average 24 hour range				22.7
Area 7 **Iona Island WWTP**	AM Low	52.5	137.0	84.5	111.6
	Noon Peak	55.0	110.0	55.0	75.0
	PM Peak	47.5	85.0	37.5	65.6
	Average 24 hour range				46.0

Note:

1) One data point discarded in average calculations (400 mg/L – August 20, 2003) as value significantly greater than other AM Low values.
2) Detection limit of 2.5 mg/L.

i) Chloride concentration was measured as a tracer to monitor effluent streams. In monitoring the chloride concentrations along the sewer system, the conditions of wastewater inflow and groundwater infiltration can be determined through a mass balance exercise.

ii) A large range and variance in the chloride readings was noted. Generally values were within accepted normal average values for chloride of approximately 80 mg/L, with the exception of the 400 mg/L recorded for the AM Low period in Area 6 on August 20, 2003. This value was considered invalid and not used in average chloride concentration.

iii) This range cannot be considered unusual due to the varied nature of the effluent stream and sampling method.

iv) The average Chloride concentration was found to be higher during the AM Low period and decrease during the peak periods.

v) The AM Low period consistently had the highest average Chloride concentration (68.3 – 111.6 mg/L).

vi) The Noon Peak period had average Chloride concentrations lower than the AM Low period and similar to the PM Peak period (45.6 – 75.0 mg/L)

vii) The average PM Peak period Chloride concentrations were similar to the Noon Peak period and lower than the AM Low period results (57.5 – 65.6 mg/L).

viii) When reviewed over time and in series this indicator does not highlight any unknown influent streams.

ix) The higher Chloride content recorded in Area 7 during the AM Low and Noon Peak periods can be attributed to the additional influent streams located directly at the IIWWTP. These streams include the Trucked Liquid Waste, Vancouver Airport Sewage, Thickener Return, Screenings Return, and Lagoon Return.

4.5 pH

Samples were measured for pH as the effluent was drawn using the Hach EC20 Portable pH/ISE Meter Model 50075. The following is a summary of these results by area:

TABLE 4.9
SUMMARY OF PH RESULTS

Location	Flow Period	Minimum pH	Maximum pH	Range	Average pH
Area 2 **8th Avenue Interceptor**	AM Low	7.01	7.18	0.17	7.11
	Noon Peak	7.05	7.42	0.37	7.18
	PM Peak	6.41	6.90	0.49	6.66
	Average 24 hour range				0.52
Area 3 **Highbury Trunk Sewer**	AM Low	6.52	6.86	0.34	6.71
	Noon Peak	6.98	7.27	0.29	7.18
	PM Peak	6.88	7.19	0.31	6.99
	Average 24 hour range				0.47
Area 6 **North Arm Interceptor**	AM Low	6.64	7.01	0.37	6.83
	Noon Peak	7.19	7.38	0.19	7.29
	PM Peak	6.36	6.82	0.46	6.65
	Average 24 hour range				0.64
Area 7 **Iona Island WWTP**	AM Low	6.81	7.09	0.28	6.97
	Noon Peak	6.91	7.14	0.23	7.05
	PM Peak	6.61	6.93	0.32	6.79
	Average 24 hour range				0.26

Note:
1) Measurement accuracy ± 0.005.

i) As summarized above, the average pH increased from the AM Low to the AM/PM Peak and then fell during the PM Peak period.

ii) The AM Low period generally had an average pH level lower than the Noon Peak period and higher than the PM Peak period (6.71 – 7.11).

iii) The Noon Peak period consistently had the highest average pH level (7.05 – 7.29)

iv) The PM Peak period generally had the lowest average pH level (6.65-6.99). This was consistent for all areas except for Area 3 where the average low was during the AM Low period.

v) The range of pH was found to be consistent between areas and is within the neutral band typical for sewage effluent in Vancouver.

4.6 TEMPERATURE

Temperature readings were taken for each sample as the effluent was drawn using the Hach EC20 Portable pH/ISE Meter Model 50075. The following provides a summary of these results by area:

TABLE 4.10
SUMMARY OF TEMPERATURE RESULTS

Location	Flow Period	Minimum Temp. (°C)	Maximum Temp. (°C)	Range (°C)	Average Temp. (°C)
Area 2 **8th Avenue Interceptor**	AM Low	20.7	21.5	0.8	21.0
	Noon Peak	22.1	22.5	0.4	22.3
	PM Peak	22.2	22.4	0.2	22.3
	Average 24 hour range				1.3
Area 3 **Highbury Trunk Sewer**	AM Low	21.3	21.6	0.3	21.4
	Noon Peak	23.7	24.4	0.7	24.0
	PM Peak	22.7	23.0	0.3	22.9
	Average 24 hour range				2.6
Area 6 **North Arm Interceptor**	AM Low	19.5	20.2	0.7	19.9
	Noon Peak	21.1	21.6	0.5	21.3
	PM Peak	21.0	21.3	0.3	21.2
	Average 24 hour range				1.4
Area 7 **Iona Island WWTP**	AM Low	21.2	21.4	0.2	21.4
	Noon Peak	22.6	22.7	0.1	22.7
	PM Peak	22.4	22.8	0.4	22.6
	Average 24 hour range				1.3

Note:
1) Measurement accuracy ± 1.0°C.

i) The measured temperature was consistent at the preselected monitoring times over the sampling period.

ii) Generally the temperature rose from the AM Low period to the AM/PM Peak period and then began to fall again towards the PM Peak period.

iii) The AM Low period consistently had the lowest average temperature (19.9 – 21.4 °C).

iv) The Noon Peak period consistently had the highest average temperature (21.3 – 24.0 °C)

v) The average PM Peak period temperatures were lower or equal to the Noon Peak period but higher than the AM Low period results (21.2 – 22.9 °C).

vi) Results where, overall, similar at the monitoring sites with the exception of Area 2 showing less of a temperature decrease from AM/PM Peak period to PM Peak period and Area 7 having the lowest range in temperature.

vii) The recorded temperature is within that expected for sewage effluent at these locations.

5 OPTIONS FOR IN-SEWER TREATMENT

The purpose of this section is to identify options for improving dissolved oxygen levels and toxicity test results in the plant effluent. Several options were considered for in-sewer corrective action to reduce organic carbon concentrations received at Iona Island WWTP, or to inhibit oxygen resources depletion during the bioassay tests performed on primary effluent. Some of these are rather obviously too expensive or inherently ineffective but are included in the following discussion for completeness.

5.1 TOXICITY REDUCTION BY IN-SEWER AERATION

A strategy proposed to control effluent toxicity from the primary treatment plants at IIWWTP and Lion's Gate Wastewater Treatment Plant (LGWWTP) is to accelerate the degradation of soluble organics in the major trunk sewers such as the Highbury tunnel. This would consist of increasing the dissolved oxygen levels in the wastewater flow by diffusing compressed air into the flow at several locations along the length of the Highbury trunk so that the concentration of DO in the sewage flow would be well above 1.0 mg/L . This level of dissolved oxygen would then encourage the activity of micro-organisms suspended in the sewage flow or present as a bio-film on the wetted surface of the sewer invert. Theoretically the increased organism growth rates would tend to deplete readily degradable organic materials arriving at the plant and subsequently in the primary effluent. If the readily degradable organics are decreased then the residual organics and organisms following primary treatment may not cause as great a level of oxygen depletion during the toxicity batch tests.

A significant amount of research has been carried out to assess the potential for BOD and COD reduction in aerated sewers. e.g. Pomeroy and Parkhurst, " Self Purification in Sewers", Proc. 6th International Water Pollution Research Symposium, 1972. and Ozer and Kasirga, "Substrate Removal in Long Sewer Lines" Water Science Technology, 1995. In this latter paper, which summarized research on this topic, it showed that for large diameter sewers of 200 cm and larger, flowing half full at about 0.5m/sec, the reduction in initial BOD concentration through suspended growth activity could be between 12 and 24 % in a sewer length of 3.5 to 7.5 km. This level of soluble BOD reduction could then be expected to occur in the Highbury and the Eighth Avenue Interceptors if aeration were installed. Equations are provided in this paper to calculate organic load reduction as a function of sewer length and diameter.

There is a major question as to the whether such a strategy would cause a decrease or an increase in the oxygen depletion occurring in batch tests on primary effluent. The growth of micro-organisms stimulated by in-sewer aeration might be so successful that micro-organism levels present following primary treatment would more rapidly use the oxygen supplied initially or during the test than current experience without sewer aeration.

The small scale testing carried out as part of the Iona and Lion's Gate Facility plan has shown, that in order to reduce the toxicity resulting from oxygen depletion occurring during toxicity tests, the soluble BOD must be reduced by about 50% to 70%.

Therefore, to achieve toxicity reduction through biological activity in the sewers would require a more sophisticated treatment system than aeration. A means of concentrating the suspended growth and sludge recycling along the length of the sewer would have to be provided; essentially a subsurface activated sludge or fixed film process. The practicality of achieving this is therefore questionable and the costs would be more expensive than an end of the pipe treatment upgrade at Iona or Lion's Gate treatment plants.

The magnitude of the blowers and air diffusion system to provide aeration in the Highbury Interceptor would be significant requiring approximately 25,000 kg/d of oxygen to achieve 20% reduction in soluble organics (3,600,000 m^3/d of compressed air, assuming 5% of standard oxygen transfer efficiency at 100 cm sewer depth) for the ADWF of 500,000 m^3/day of wastewater flow.

There is no doubt that installation of an aeration system into the major trunk sewers such as the Highbury Tunnel would raise the dissolved oxygen levels arriving at the Iona Island WWTP probably well above 2.0 mg/L. However the concentration of DO arriving at the plant or even present in the primary effluent is not the cause of the toxicity occurring in the tests. It is the combination of the presence of high concentrations of easily biodegradable organics and the presence of aerobic micro-organisms in the primary effluent which are the major cause of the measured primary effluent acute toxicity.

5.2 TOXICITY REDUCTION BY CHEMICAL ADDITION

Another strategy to reduce the soluble organics or MBAS levels in primary effluent would be to add sufficient strongly oxidizing chemical such as hydrogen peroxide to achieve the 50% to 70% reduction in soluble organics associated with toxicity reduction as shown by the small scale testing carried out as part of this project. This would mean that for a soluble organic load reduction of 25,000 kg/d (50% soluble BOD or 20% of total BOD) during the ADWF period, an equivalent hydrogen peroxide oxidizing potential to the biological removal oxygen requirement would need to be applied at appropriate locations along the length of the Highbury Interceptor. Considering the relative oxidizing potential of a chemical such as hydrogen peroxide compared to oxygen the amount of hydrogen peroxide for the ADWF loads would be approx. 70,000 kg/d. This is a significant amount of chemical and at $126,000/d (50% H_2O_2 solution) for hydrogen, this would represent a prohibitively expensive operating cost. Toxicity reduction by chemical addition to oxidize soluble organics and MBAS is impractical from an operational cost viewpoint.

5.3 TOXICITY REDUCTION BY DISINFECTION

Another options is to kill the organisms that are transported to the plant by addition of a strong disinfection agent such as chlorine, ozone or hydrogen peroxide. Microbiological examination of samples of raw wastewater and primary effluent carried out at the time of sample collection for toxicity analysis by the GVRD operators has reportedly shown the presence of organisms and colonies resembling activated sludge organisms. Apparently toxicity testing of unchlorinated primary effluent as compared to chlorinated primary effluent samples has shown that, unchlorinated samples were toxic whereas chlorinated samples were not toxic or less toxic. The conclusions drawn by the GVRD staff were that chlorination killed, or sufficiently attenuated the micro-organisms contained in the primary effluent samples, to reduce oxygen depletion during the LC_{50} testing to such an extent that acute toxicity of the samples was reduced.

The results of the impact of chlorination on attenuating biological activity and reducing toxicity by chlorination were reported in an IRC Integrated Resource Consultant Inc. report entitled "Chlorination Treatment Trials with Iona WWTP Final Effluent". Chlorine doses varying from 2 to 12 mg/L were added to primary effluent samples and retained in contact prior to de-chlorination for periods of 15 minutes to 4 hours. Subsequent batch toxicity testing showed that at the higher chlorine dosage levels of 4 to 12 mg/L and longer detention times acute toxicity of the primary effluent samples was reduced. In some cases, the mortality of test organisms was reduced such that the chlorinated samples were in compliance with the 96 hour LC_{50} tests. In others the mortality was simply decreased in comparison to un-chlorinated samples.

As part of the small scale testing carried out (See Appendix # 5), primary effluent samples obtained as grab samples during the ADWF (low flow conditions) at a time of the day when MBAS and soluble organics were known to be high, were also treated at a sufficiently high chlorine dose to kill or attenuate any inflowing micro-organisms. Chlorine dosage was 2.0 mg/L and the samples were held for 2 hours prior to dechlorination. All three of the samples chlorinated and de-chlorinated were toxic as were the non-disinfected primary effluent samples. Although this was a very small sample size we conclude that disinfection with chlorine or another disinfectant chemicals at levels normally experienced at WWTP's would not reduce oxygen depletion during the toxicity test period of 96 hours sufficiently to be considered as a viable method to help avoid toxicity non compliance.

In any event the Stantec tests provided sufficient evidence that simple disinfection consistent with usual WWTP chlorination practice, would not insure compliance with the toxicity test criteria. GVRD testing showed that higher dosage chlorination and detention times provide toxicity reduction and in most cases acute toxicity compliance. However; at the elevated chlorination and detention time levels there is an increased probability of formation of chlorinated organics which would need to be determined and assessed with respect to their effect on the environment.

The GVRD have that rather than chlorinating the primary effluent and retaining it for an extended contact time, it might make more economic sense to chlorinate into the sewer system at a location in the Highbury Interceptor. This would avoid the cost of constructing chlorine contact tank. Because the raw sewage is at least 30 % higher in organic strength than primary effluent, chlorine dosage rates would have to be even higher than the GVRD test dosages on primary effluent to achieve the same bacteria kill and toxicity reduction during the tests. There would therefore be an even higher possibility of formation of undesirable chlorinated organics.

6 SUMMARY AND RECOMMENDATIONS

It was postulated that toxicity reduction at IIWWTP could be significantly influenced by actions taken in the sewer system. The toxicity problem at Iona has been identified as being caused primarily by oxygen depletion occurring during the compliance monitoring toxicity testing. Oxygen depletion during the test is caused by the presence of readily degradable organics in the primary effluent as well as the action of microorganisms present in the primary effluent. GVRD personnel have noted the presence of organisms, similar to activated sludge organisms, in the samples sent to the toxicity-testing laboratory at times when the toxicity tests have failed.

Strategies to eliminate and reduce the occurrence of toxicity test failures by reducing the source of soluble organics and that have been proposed are described hereafter:

- By aeration of a portion of Highbury Tunnel and stimulation of aerobic biological growth and degradation of soluble organics
- By adding oxidizing chemicals into the sewer system such as hydrogen peroxide to chemically degrade organics
- Suppress the growth of microorganisms in the sewers so that their population in the primary effluent is sufficiently low such that their respiration during the toxicity tests does not deplete the DO to levels that kill the test fish
- By adding a strong oxidizing agent to the sewers such as chlorine.
- Controlling the development of soluble organics resulting from the solubilization of from settled organic solids.
- By changing the flow regime and eliminating sludge accumulations by flushing the pipes are regular intervals.

If such strategies were feasible, the cost may be significantly less than providing interim treatment at the Iona plant.

As part of this study, two activities have provided information on how effective in-sewer control activities might be. The first activity which was carried out as part of Appendix 2 included field sampling for soluble and total organics and dissolved oxygen, was completed at three locations of the Highbury Interceptor and in tributary inputs to this major line. Modeling of sewer system using the hydraulic component of the DHI Mouse Trap model was carried out to calculate flow velocities at average dry weather flow conditions at key locations along the sewer lines. As well the Mouse Trap Model water quality components were used to develop mass balances of key parameters such as COD .

The second activity, which was carried out as part of Appendix 5 including batch chemically enhanced, primary, and biological treatment was undertaken in August of 2003 during the dry weather flow period treating both the raw and primary effluent at both Iona and Lions Gate Wastewater Treatment Plants. Standard LC_{50} toxicity testing as well as testing for total and soluble BODs, TSS, and surfactants (MBAS methylene blue active substances) was carried out.

The field sampling and testing in the sewer system were limited in nature but did show the following:

- Little trending in soluble organics or TSS occurred in the major interceptor sewer from upstream to downstream sections of the Highbury interceptor other than an expected increase in organic, and solid load, consistent with increased inputs along the trunk sewer.
- Flow velocity calculations from the DHI model indicated that, even during average low flow conditions, the velocities in the main trunks did not decrease to levels where organics would settle out into the invert of the sewers.
- Throughout the trunk sewer system sampled, the dissolved oxygen levels were generally less than 1 mg/L.

From this information, it appears that there are microorganisms at work in the sewers that are utilizing the available dissolved oxygen but these are not significantly reducing the organic loading.

The six sets of small-scale treatment batch tests provided good information on the extent of organics and surfactants removal that has to be achieved to obtain a significant reduction in toxicity. To reduce the frequency of occurrence of acute toxicity, at least 100% chemically enhanced primary or 50% biological treatment (100% of load receiving primary settling plus 50% of ADWF biological treatment) has to be carried out. The required extent of soluble organics removal to improve the LC_{50} test results appears to be 52 to 77%. This is a significant reduction in organics.

The following conclusions can be made:

- Controlling toxicity by reducing the industrial organics load would not be feasible because at the Iona Island plant, the total industrial load only represents about 15% of the total BOD load to the plant. Forcing pretreatment on the industry by a change in by-law would not be successful in reducing toxicity because a 50 to 77% reduction in mass loading is not physically possible through industrial load reduction.
- Similarly, to achieve a mass reduction in soluble BOD in the range of 50 to 77% would mean converting a portion of the sewer system into a biological or chemical treatment facility. The addition of chemical oxidants such as hydrogen peroxide, potassium permanganate, or ferric salts could not achieve that level of organic destruction at a reasonable operating cost. Creating an in-sewer, tubular reactor biological treatment system would require the equivalent, or greater, capital cost than partial biological (50% ADWF) treatment at the Iona plant. We are not aware of a major application of these techniques in North America. Transport of the biological solids generated by such an in-sewer system would also be problematic.
- Addition of a chemical agent such as chlorine to lower the level of viable micro-organisms to the point where primary effluent contains such low levels of aerobic organisms could be successful at high chlorine doses and retention times. However, formation of chlorinated organics and their associated environmental impacts would need to be considered

- In-sewer treatment is not a feasible option to reduce primary effluent toxicity and until full secondary treatment is implemented at Iona Island WWTP interim upgrades at the plant are required in order to achieve significant improvement in LC50 test results.

Considering the information reported in this memo, based on the finding of the field-testing, monitoring and the small-scale treatment for toxicity reduction, it would not be fruitful to pursue detailed trunk sewer system water quality modeling using the DHI Mouse Trap model. The in-sewer system strategies available to the GVRD as discussed above would not be effective in reducing toxicity to compliance levels.

APPENDIX A: ANALYTICAL RESULTS

BCAT British Columbia Analytical Technologies Ltd.

August 24, 2003

Jowitt Li
Suite 1007, 7445 132 Street
Surrey, BC V3W 1J8

Dear Dr. Li:

Please find the attached BCAT analytical reports of A0308007S51, S0308008S52 and S0308009S53. These reports are for the water samples received on August 13, 14 and 16, 2003 respectively. Please feel free to contact me should you have any questions.

Sincerely,

Parker Shieh

Parker Shieh, Ph.D.
Supervisor, Environmental Services

120A-3989 Henning Drive, Burnaby, BC V5C 6N5 Canada • Tel: 604-320-1588 • Fax: 604-434-9577

British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308007S51
FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8
Attn: Jowitt Li

Project: NA
Date Received: August 13, 2003
Date Completed: August 21, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
3-1A	Aug-13-03	Water	400	-
3-1B	Aug-13-03	Water	-	65.0
2-1A	Aug-7-03	Water	280	-
2-1B	Aug-13-03	Water	-	120
6-1A	Aug-13-03	Water	170	-
6-1B	Aug-13-03	Water	-	55.0
7-1A	Aug-13-03	Water	420	-
7-1B	Aug-13-03	Water	-	110

Analytical Results Certified by:

Dr. Parker Shieh: Director, Environmental Services

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Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Chemical Oxygen Demand	Closed Reflux/Colorimetric APHA5220 D	20	201
Chloride	Filtration/Incubation/DO APHA5210 B 4500-OGC	2.5	301

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Chemical Oxygen Demand

Measure an appropriate amount of each sample and reagents into a tube. Prepare the sample, blank, and standards into culture tubes followed by digestion and let them cool thereafter. Invert cooled samples, blank, and standards several times and allow solids to settle before the colorimetric measurement for absorbance at the wavelength of 600 nm. Prepare at least five standards from potassium hydrogen phthalate solution with COD equivalents from 20 to 900 mg oxygen per litre. Make up to volume with D.I. water through the same digestion process as for samples. The results of the samples are determined according to the calibration curve of the standard (APHA 5220D).

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Chloride

A aliquot of well-mixed sample is measured followed by adding indicator. The sodium chloride standard solution is then prepared for the calibration. The sample is titrated by 0.0493 N of silver nitrate titration solution. The chloride concentration is determined by titration calculation.

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BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308008S52

FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8

Attn: Jowitt Li

Project: NA
Date Received: August 14, 2003
Date Completed: August 21, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
3-2A	Aug-14-03	Water	350	-
3-2B	Aug-14-03	Water	-	45.0
2-2A	Aug-14-03	Water	290	-
2-2B	Aug-14-03	Water	-	40.0
6-2A	Aug-14-03	Water	200	-
6-2B	Aug-14-03	Water	-	45.0
7-2A	Aug-14-03	Water	270	
7-2B	Aug-14-03	Water	-	47.5
3-3A	Aug-14-03	Water	58	-
3-3B	Aug-14-03	Water	-	42.5

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
2-3A	Aug-14-03	Water	110	-
2-3B	Aug-14-03	Water	-	62.5
6-3A	Aug-14-03	Water	47	-
6-3B	Aug-14-03	Water	-	112.5
7-3A	Aug-14-03	Water	110	-
7-3B	Aug-14-03	Water	-	52.5
3-4A	Aug-14-03	Water	360	-
3-4B	Aug-14-03	Water	-	70.0
2-4A	Aug-14-03	Water	340	-
2-4B	Aug-14-03	Water	-	62.5
6-4A	Aug-14-03	Water	170	-
6-4B	Aug-14-03	Water	-	35.0
7-4A	Aug-14-03	Water	420	-
7-4B	Aug-14-03	Water	-	77.5

Analytical Results Certified by: Parker Shieh

Dr. Parker Shieh: Director, Environmental Services

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Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Chemical Oxygen Demand	Closed Reflux/Colorimetric APHA5220 D	20	201
Chloride	Filtration/Incubation/DO APHA5210 B 4500-OGC	2.5	301

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Chemical Oxygen Demand

Measure an appropriate amount of each sample and reagents into a tube. Prepare the sample, blank, and standards into culture tubes followed by digestion and let them cool thereafter. Invert cooled samples, blank, and standards several times and allow solids to settle before the colorimetric measurement for absorbance at the wavelength of 600 nm. Prepare at least five standards from potassium hydrogen phthalate solution with COD equivalents from 20 to 900 mg oxygen per litre. Make up to volume with D.I. water through the same digestion process as for samples. The results of the samples are determined according to the calibration curve of the standard (APHA 5220D).

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Chloride

A aliquot of well-mixed sample is measured followed by adding indicator. The sodium chloride standard solution is then prepared for the calibration. The sample is titrated by 0.0493 N of silver nitrate titration solution. The chloride concentration is determined by titration calculation.

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BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308009S53
FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8
Attn: Jowitt Li

Project: NA
Date Received: August 15, 2003
Date Completed: August 24, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
3-5A	Aug-14-03	Water	330	-
3-5B	Aug-14-03	Water	-	50.0
2-5A	Aug-14-03	Water	340	-
2-5B	Aug-14-03	Water	-	47.5
6-5A	Aug-14-03	Water	240	-
6-5B	Aug-14-03	Water	-	20.5
7-5A	Aug-14-03	Water	320	
7-5B	Aug-14-03	Water	-	62.5
3-6A	Aug-14-03	Water	170	-
3-6B	Aug-14-03	Water	-	75.0

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
2-6A	Aug-14-03	Water	79	-
2-6B	Aug-14-03	Water	-	37.5
6-6A	Aug-14-03	Water	110	-
6-6B	Aug-14-03	Water	-	60.0
7-6A	Aug-14-03	Water	140	-
7-6B	Aug-14-03	Water	-	125

Analytical Results Certified by: Parker Shieh

Dr. Parker Shieh: Director, Environmental Services

120A-3989 Henning Drive, Burnaby, BC V5C 6N5 Canada • Tel: 604-320-1588 • Fax: 604-434-9577

Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Chemical Oxygen Demand	Closed Reflux/Colorimetric APHA5220 D	20	201
Chloride	Colorimetric/Silver Nitrate EPA SW 846 - 9253	2.5	301

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Chemical Oxygen Demand

Measure an appropriate amount of each sample and reagents into a tube. Prepare the sample, blank, and standards into culture tubes followed by digestion and let them cool thereafter. Invert cooled samples, blank, and standards several times and allow solids to settle before the colorimetric measurement for absorbance at the wavelength of 600 nm. Prepare at least five standards from potassium hydrogen phthalate solution with COD equivalents from 20 to 900 mg oxygen per liter. Make up to volume with D.I. water through the same digestion process as for samples. The results of the samples are determined according to the calibration curve of the standard solutions (APHA 5220D).

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Chloride

This method is intended for the chloride content is 5 mg/L or more and where interferences such as color or high concentrations of heavy metal ions present in the sample. An aliquot of well-mixed sample is measured followed by adding potassium chromate indicator. The sodium chloride standard solution is then prepared for the calibration. The sample is titrated by 0.0493 N of silver nitrate titration solution. The chloride concentration is determined by titration calculation.

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BCAT British Columbia Analytical Technologies Ltd.

QUALITY CONTROL SUMMARY

(A0308007S51QC, A0308008S52QC and A0308009S53QC)

Parameter: Chemical Oxygen Demand

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD	Notes
Blank 1	< 20	< 20	-	-	-
Blank 2	< 20	< 20	-	-	-
Blank 3	< 20	< 20	-	-	-
Standard 1	478	500	96	-	-
Standard 2	494	500	99	-	-
Standard 3	500	500	100	-	-
Duplicate 1	187	170	-	10	6-1A
Duplicate 2	63	58	-	-	3-3A
Duplicate 3	171	170	-	0.6	6-4A

- RPD: Relative Percent Difference (Acceptable RDP is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

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Parameter: Chloride

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD %	Notes
Blank 1	< 2.5	< 2.5	-	-	-
Blank 2	2.5	< 2.5	-	-	-
Blank 3	2.5	< 2.5			
Standard 1	100	105	95	-	-
Standard 2	105	105	100	-	-
Standard 3	103	105	98	-	-
Duplicate 1	112.5	110	2.2	-	7-1B
Duplicate 2	207	205	1.0	-	6-5B
Duplicate 3	82.5	85.0	3.0	-	7-7B

* RPD: Relative Percent Difference (Acceptable RDP is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

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British Columbia Analytical Technologies Ltd.

QUALITY CONTROL SUMMARY

(Revised)

(A0308007S51QC, A0308008S52QC and A0308009S53QC)

Parameter: Chemical Oxygen Demand

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD %	Notes
Blank 1	< 20	< 20	-	-	-
Blank 2	< 20	< 20	-	-	-
Blank 3	< 20	< 20	-	-	-
Standard 1	478	500	96	-	-
Standard 2	494	500	99	-	-
Standard 3	500	500	100	-	-
Duplicate 1	187	170	-	10	6-1A
Duplicate 2	63	58	-	-	3-3A
Duplicate 3	171	170	-	0.6	6-4A

- RPD: Relative Percent Difference (Acceptable RPD is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

Parameter: Chloride

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD %	Notes
Blank 1	< 2.5	< 2.5	-	-	-
Blank 2	2.5	< 2.5	-	-	-
Blank 3	2.5	< 2.5	-	-	-
Standard 1	100	105	95	-	-
Standard 2	105	105	100	-	-
Standard 3	103	105	98	-	-
Duplicate 1	112.5	110	-	2.2	7-1B
Duplicate 2	207	205	-	1.0	6-5B
Duplicate 3	82.5	85.0	-	3.0	7-7B

* RPD: Relative Percent Difference (Acceptable RDP is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

BCAT A0308007S51QC/A0308008S52QC/A0308009S53QC

BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #: A0308014S58
FOR: Stantec Consulting
Suite 1007, #7445, 132Street
Surey, B.C.
V3W 1J8
Attn: Jowitt Li

Project: NA
Date Received: August 20, 2003
Date Completed: August 25, 2003
Fax #: 604-591-1856
Phone #: 604-597-0442
Account: HOY

Analytical Results:

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
3-7A	Aug-14-03	Water	320	-
3-7B	Aug-14-03	Water	-	80.0
2-7A	Aug-14-03	Water	320	-
2-7B	Aug-14-03	Water	-	70.0
6-7A	Aug-14-03	Water	190	-
6-7B	Aug-14-03	Water	-	62.5
7-7A	Aug-14-03	Water	400	
7-7B	Aug-14-03	Water	-	85.0
3-8A	Aug-14-03	Water	420	-
3-8B	Aug-14-03	Water	-	112.5

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
2-8A	Aug-14-03	Water	74	-
2-8B	Aug-14-03	Water	-	102
6-8A	Aug-14-03	Water	130	-
6-8B	Aug-14-03	Water	-	400
7-8A	Aug-14-03	Water	260	-
7-8B	Aug-14-03	Water	-	132
3-9A	Aug-14-03	Water	360	-
3-9B	Aug-14-03	Water	-	60.0
2-9A	Aug-14-03	Water	260	-
2-9B	Aug-14-03	Water	-	47.5
6-9A	Aug-14-03	Water	260	-
6-9B	Aug-14-03	Water	-	50.0
7-9A	Aug-14-03	Water	430	-
7-9B	Aug-14-03	Water	-	57.5

Analytical Results Certified by: ______________________

Dr. Parker Shieh: Director, Environmental Services

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Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Chemical Oxygen Demand	Closed Reflux/Colorimetric APHA5220 D	20	201
Chloride	Colorimetric/Silver Nitrate EPA SW 846 - 9253	2.5	301

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Chemical Oxygen Demand

Measure an appropriate amount of each sample and reagents into a tube. Prepare the sample, blank, and standards into culture tubes followed by digestion and let them cool thereafter. Invert cooled samples, blank, and standards several times and allow solids to settle before the colorimetric measurement for absorbance at the wavelength of 600 nm. Prepare at least five standards from potassium hydrogen phthalate solution with COD equivalents from 20 to 900 mg oxygen per litre. Make up to volume with D.I. water through the same digestion process as for samples. The results of the samples are determined according to the calibration curve of the standard (APHA 5220D).

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Chloride

This method is intended for the chloride content is 5 mg/L or more and where interferences such as color or high concentrations of heavy metal ions present in the sample. An aliquot of well-mixed sample is measured and followed by adding potassium chromate indicator. The sodium chloride standard solution is then prepared for the calibration. The sample is titrated by 0.0493 N of silver nitrate titration solution. The chloride concentration is determined by titration calculation.

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BCAT British Columbia Analytical Technologies Ltd.

ANALYTICAL REPORT

CERTIFICATE #:	A0308015S59	*Project:*	NA
FOR:	Stantec Consulting	*Date Received:*	August 21, 2003
	Suite 1007, #7445, 132Street	*Date Completed:*	August 25, 2003
	Surey, B.C.	*Fax #:*	*604*-591-1856
	V3W 1J8	*Phone #:*	604-597-0442
Attn:	Jowitt Li	*Account:*	*HOY*

Analytical Results:

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
3-10A	Aug-14-03	Water	320	-
3-10B	Aug-14-03	Water	-	55.0
2-10A	Aug-14-03	Water	280	-
2-10B	Aug-14-03	Water	-	62.5
6-10A	Aug-14-03	Water	200	-
6-10B	Aug-14-03	Water	-	122
7-10A	Aug-14-03	Water	360	-
7-10B	Aug-14-03	Water	-	67.5
3-11A	Aug-14-03	Water	180	-
3-11B	Aug-14-03	Water	-	67.5

Client Sample Identification	Sample Date	Matrix	Chemical Oxygen Demand (mg/L)	Chloride (mg/L)
2-11A	Aug-14-03	Water	160	-
2-11B	Aug-14-03	Water	-	140
6-11A	Aug-14-03	Water	56	-
6-11B	Aug-14-03	Water	-	32.5
7-12A	Aug-14-03	Water	210	-
7-12B	Aug-14-03	Water	-	137
3-12A	Aug-14-03	Water	340	-
3-12B	Aug-14-03	Water	-	65.0
2-12A	Aug-14-03	Water	310	-
2-12B	Aug-14-03	Water	-	57.5
6-12A	Aug-14-03	Water	190	-
6-12B	Aug-14-03	Water	-	42.5
7-12A	Aug-14-03	Water	440	-
7-12B	Aug-14-03	Water	-	55.0

Analytical Results Certified by: ______________________

Dr. Parker Shieh: Director, Environmental Services

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Summary of Analytical Methods

Parameter	Analytical Method	RDL (mg/L)*	Lab Code
Chemical Oxygen Demand	Closed Reflux/Colorimetric APHA5220 D	20	201
Chloride	Colorimetric/Silver Nitrate EPA SW 846 - 9253	2.5	301

* The reported detection limits is based on the standard condition of samples .The detection limits are varied depending on the dilution factor.

General Comments:

- All solids results are reported on a dry weigh basis unless otherwise noted.
- Units: mg/kg (milligrams per Kilograms, equivalent to parts per million, ppm)
  mg/L (milligrams per Litre, equivalent to parts per million, ppm)
  ug/L (microgram per Litre, equivalent to parts per billion, ppb)
- "RDL": Reported Detection Limit.
- "<": Less than reported detection Limit.

Brief Method Description:

Chemical Oxygen Demand

Measure an appropriate amount of each sample and reagents into a tube. Prepare the sample, blank, and standards into culture tubes followed by digestion and let them cool thereafter. Invert cooled samples, blank, and standards several times and allow solids to settle before the colorimetric measurement for absorbance at the wavelength of 600 nm. Prepare at least five standards from potassium hydrogen phthalate solution with COD equivalents from 20 to 900 mg oxygen per litre. Make up to volume with D.I. water through the same digestion process as for samples. The results of the samples are determined according to the calibration curve of the standard (APHA 5220D).

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Chloride

This method is intended for the chloride content is 5 mg/L or more and where interferences such as color or high concentrations of heavy metal ions present in the sample. An aliquot of well-mixed sample is measured and followed by adding potassium chromate indicator. The sodium chloride standard solution is then prepared for the calibration. The sample is titrated by 0.0493 N of silver nitrate titration solution. The chloride concentration is determined by titration calculation.

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QUALITY CONTROL SUMMARY

(A0308014S58QC and A0308015S59QC)

Parameter: Chemical Oxygen Demand

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD %	Notes
Blank 1	< 20	< 20	-	-	-
Blank 2	< 20	< 20	-	-	-
Blank 3	< 20	< 20	-	-	-
Standard 1	499	500	100	-	-
Standard 2	489	500	98	-	-
Standard 3	517	500	103	-	-
Duplicate 1	353	320	-	10	3-7A
Duplicate 2	1780	1800	-	1	OB
Duplicate 3	317	320	-	0.9	3-10A

- RPD: Relative Percent Difference (Acceptable RPD is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

Parameter: Chloride

QC Sample	Result (mg/L)	Target Value (mg/L)	Recovery %	RPD %	Notes
Blank 1	< 2.5	< 2.5	-	-	-
Blank 2	2.5	< 2.5	-	-	-
Blank 3	2.5	< 2.5	-	-	-
Standard 1	105	105	100	-	-
Standard 2	100	105	95	-	-
Standard 3	105	105	100	-	-
Standard 4	103	105	98	-	-
Duplicate 1	112.5	110	-	2.2	7-1B
Duplicate 2	82.5	85.0	-	3.0	7-7B
Duplicate 3	62.5	62.5	-	0.0	2-10B

* RPD: Relative Percent Difference (Acceptable RPD is set for no more than 20% and will not be calculated where either duplicate or source sample result is less than 5X of RDL)

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GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 3
Interim Treatment Facility Upgrading Requirements

FINAL REPORT

Prepared for

Greater Vancouver Regional District

Prepared by

Stantec Consulting Ltd.
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Surrey, BC
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August 2005

117-00018

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1 INTRODUCTION

This report (Appendix 3) describes the requirements for the interim upgrades to the Iona Island and Lions Gate wastewater treatment plants. The interim upgrades are designed to be in place until both plants are converted to secondary treatment facilities. The build-out to secondary must occur by 2020 for Iona Island WWTP and by 2030 for Lions Gate WWTP.

This report should be read in conjunction with Appendix 4, which describes the requirements for build-out to secondary.

Appendix 3 and 4 should also be read in conjunction with Appendix 10. Essentially Appendix 10 is the continuation of Appendix 3 and 4. Appendix 3 and 4 describe the analysis for the first and the second levels of screening where the number of options for interim upgrades was reduced from 27 to 4 or 5 and the number of options for upgrade to secondary was reduced from 14 to 1 or 2.

The short list of options identified in Appendix 3 and 4 are then examined in more detail in Appendix 10 including revised cost estimates. The cost estimates in Appendix 3 and 4 were developed as tools for the screening of options. For more accurate cost estimates, the reader should refer to Appendix 10.

Also, while the work covered by Appendix 3 and 4 was under way, the reports for Appendix 7 (Interim Sludge Handling) and Appendix 8 (Condition of Existing Treatment Plant) were being developed. The recommendations of the reports for Appendix 7 and 8 were carried over in the report for Appendix 10.

Finally, the reports for Appendix 1 to 10 are brought together in the Summary Report.

The effluent criteria for the interim upgrades were formulated as part of the Liquid Waste Management Plan and these are described in Section 2. The base level of treatment that must be provided to meet effluent criteria for BOD and TSS is presented. In order to determine how the various treatment upgrade options could improve effluent quality and improve toxicity test results, small scale bench testing was carried out. The results of the small scale testing are summarized in Section 3.5. Complete details regarding the small scale-testing program are presented separately in Appendix 5.

Detailed flow and load projections were carried out in order to generate a lower and upper envelope. Separate flow and load projections were prepared for the various contributors, including: (1) residential, (2) commercial and institutional, (3) industrial, (4) groundwater infiltration and (5) trucked liquid waste and stormwater for combined sewers tributary to Iona. For each contributor, lower and upper growth rates were established and the impact of various scenarios for source control were estimated. Lower and upper envelopes for flows and loads were prepared by adding the lower and upper envelopes for the five above components. This work is described in detail in Section 4.

Summary descriptions of the Iona Island and Lions Gate wastewater treatment plants are included in Section 5. A brief description of in-pipe or upstream treatment alternatives that were examined is provided in Section 6. These in-pipe alternatives were examined to determine if the load on the plants could be reduced.

For both plants, a comprehensive number of options for interim upgrades were identified. These include (1) physicals/chemical processes including chemically enhanced primary (CEP) treatment, (2) biological treatment for a portion of the average dry weather flow (ADWF), (3) combination of CEP and partial biological treatment, (4) dissolved air flotation and (5) combination of primary treatment and chemical oxidation. Each treatment process is described in Section 7.

All the options described in Section 7 were screened and ranked using a two-step approach. All the options were initially screened using pass or fail criteria. Those options that passed were then ranked. This first level of screening and ranking is described in Section 8.

2 OBJECTIVES OF INTERIM TREATMENT

2.1 LIQUID WASTE MANAGEMENT PLAN REQUIREMENTS

The Liquid Waste Management Plan has indicated that the base level of treatment for Iona Island and Lions Gate should meet the following maximum daily concentration levels:

	BOD (mg/L)	TSS (mg/L)
Iona Island	130	100
Lions Gate	130	130

The above concentrations are based on flow proportional 24-hr composite samples. The Liquid Waste Management Plan (LWMP) further indicates that the Lions Gate Treatment Plant will provide primary treatment for flows up to two times dry weather flow and that the Iona Island Treatment plant will provide primary treatment for flows up to 17 m^3/sec (1469 ML/d).

One of the commitments of the LWMP is to upgrade both plants by adding facilities for chemical addition if necessary in order to meet the above effluent concentrations. Another commitment of the LWMP is to construct facilities for biological treatment in order to address environmental concerns and to maintain effluent concentrations and loadings, which are beyond the capability of enhanced primary treatment.

2.2 DRAFT FEDERAL POLICY ON AMMONIA

The draft Federal policy on ammonia which was published in June 2003 applies to wastewater treatment systems where the annual average effluent release during 2004 from that system to surface water is greater than or equal to 5000 m^3 per day and where any of the following three conditions are met:

1) Residual Chlorine

   The concentration of total residual chlorine (TRC) in the release exceeded 0.02 mg/L at any time during 2004.

2) Ammonia and Depth of Outfall

   (a) The concentration of ammonia nitrogen (NH_3-N) in the effluent exceeded 16 mg/L at any time during the period of June 1, 2004, to September 30, 2004; and

   (b) The depth of water over the effluent release point, at any time during the period of June 1, 2004, to September 30, 2004, is less than 15 times the diameter of the discharge pipe or the diameter of a diffuser port in the discharge pipe.

3) Ammonia, pH and Fresh Water

   (a) The effluent release is to fresh water, and

(b) The concentration of ammonia nitrogen (NH_3-N) in the effluent exceeded 16 mg/L at any time during the period of June 1, 2004, to September 30, 2004, and

(c) The pH of the surface water upstream of the effluent release point exceeded 7.5 at any time during the period of June 1, 2004, to September 30, 2004.

Since the effluent from Iona Island and Lions Gate WWTP does not contain residual chlorine, is discharged at depths greater than 15 times the diameter of the discharge pipe and is not released to fresh water, the proposed Federal policy is not applicable.

3 EFFLUENT TOXICITY

3.1 REVIEW OF PAST TOXICITY TESTS

3.1.1 General

Regular toxicity testing of the Lions Gate and Iona Island WWTP effluents has been carried out for at least the past 10 years. As background for this current planning study, three previous reports have been reviewed – ***Effluent Toxicity Study 1997 Program, GVS&DD Wastewater Treatment Plants***, authored by GVRD; ***GVRD Liquid Waste Management Plan – Acute Toxicity Identification Evaluations of GVS&DD Wastewater Treatment Plant Effluents***, authored by EVS Environmental Consultants March 2001; and ***2001 Quality Control Annual Report for Greater Vancouver Sewerage and Drainage District – Volumes I and II***, authored by GVRD. In addition, spreadsheets containing all toxicity test results from Lions Gate and Iona Island from 1994 to May 2003 have been reviewed as part of this assignment.

The test involves placing ten rainbow trout fingerlings in varying concentrations of effluent. According to DFO and Environment Canada the only test of significance is using 100% effluent. If six or more fish die in 100% effluent, then the effluent is deemed to be toxic.

3.1.2 Iona Island Plant Effluent Toxicity

In addition to the regular effluent toxicity tests carried out once per month, there has been some special testing carried out at the Iona plant to determine the impact of low DO during the test on the degree of toxicity.

During the summers of 1999 to 2002, duplicate tests were conducted on a number of occasions to determine if the use of O_2 instead of air would enhance the poor toxicity results which appeared to be due to low DO at the beginning of, and during, the 96 hours of each test. In every duplicate test conducted the O_2 aliquot gave better results than the air aliquot (in some tests both the O_2 and air aliquots passed the test), while in all but one duplicate experiment the O_2 aliquot passed the required survival in 100% effluent.

In general, the 9 years of data show very strongly that poor toxicity test results occur almost exclusively during the low-flow period of the year.

3.1.3 Lions Gate Plant Effluent Toxicity

As with Iona Island, Lions Gate effluent toxicity tests have been carried out on a monthly basis for at least the past decade. Additionally, since ammonia toxicity has been a suspected cause of some of the poor toxicity results, special testing has been carried out with two or more aliquots of the same sample, each aliquot tested at a different pH to determine if varying free ammonia concentrations affected the toxicity results.

The general toxicity-testing program has produced results that do not indicate the low-flow effect on toxicity that was evident at Iona Island. Although there seem to be poorer toxicity results in summer than at other times of the year, there are test failures during all seasons.

The special testing at Lions Gate WWTP in 2001 involved gathering samples at various times during the day, and the simultaneous testing of 2 or 3 aliquots at differing test pH values. The purpose of the various pH tests was to determine if the concentration of free ammonia was a culprit in the toxicity non-compliance. Results indicate that little change in toxicity test results occur in samples taken at different times of the day. Similarly, varying the test pH from the normal value of about 7 to either 6 or 8 does not appear to cause any serious variation in test results.

3.2 CAUSES OF TOXICITY

3.2.1 General

Sewage treatment plant effluent aquatic species toxicity can be caused by many impurities that may be present in the plant influent. The test results at any given plant will be dependent on the concentration of toxic components in the plant influent, and the degree of removal that occurs during the specific treatment process utilized at the plant. Toxicity testing is done on grab samples from the plant effluent.

3.2.2 Iona Island Effluent

It has long been suspected by the GVRD that the main reason for the toxicity test failures on the Iona Island effluent is primarily due to the low DO conditions, which stresses the fingerlings sufficiently to cause mortality. The special testing protocols carried out by GVRD from 1999 to 2002 (see Section 3.1.1 above) provided very good proof of that hypothesis.

In their report entitled ***Acute Toxicity Identification Evaluations of GVS&DD Wastewater Treatment Plant Effluents***, EVS Consultants concluded that excessive oxygen demand from a high concentration of aerobic bacteria reduced DO during the test when the test tanks were aerated at the rate required by the test protocol, resulting in high fish mortality. There was a small amount of evidence that ammonia or hydrogen sulfphide could, on occasion, be the toxicant.

Improving the conditions that cause DO level to drop in Iona Island effluent will improve toxicity test results that occur during the low flow Summer months.

3.2.3 Lions Gate Effluent

The causes of toxicity in the Lions Gate plant effluent are apparently more varied than for Iona Island. The toxicity identification evaluation (TIE) carried out on each failed sample indicated quite strongly that the main culprit in the failures was a non-polar organic, with the most likely organic family being methylene blue active substances (MBAS). Additionally, there were 5 samples in which the TIE tests indicated that ammonia was also involved in the toxic reactions.

A visual examination of the toxicity data spreadsheet supplied by GVRD showed that since MBAS concentration began to be measured in the toxicity samples in April 2001, most of the samples which did not pass the test had MBAS concentrations above 2.5 mg/L, while those that passed the test contained less than 2.5 mg/L. The synergistic effect of ammonia could not be distinguished in the available data.

3.3 POTENTIAL STRATEGIES FOR MITIGATING TOXICITY

3.3.1 General

Given the reported results of the toxicity tests carried out at the two subject wastewater treatment plants, there are a number of mitigating short term corrective actions that could be taken to achieve better toxicity test results. The currently implicated causes of toxicity test failures at the two plants indicate that somewhat different possible short-term solutions might be considered at the Lions Gate WWTP than at Iona Island WWTP. They will therefore be discussed separately.

3.3.2 Potential Strategies at Iona Island

There are two approaches that could be taken at the Iona Island facility. The first is to alter the test conditions that allow biological activity in the aquarium to reduce the DO to the point of fish mortality, while the second is to remove sufficient BOD from the sewage to prevent the test DO from dropping so far with current test conditions.

Altering the test conditions to improve DO can take two forms: kill a sufficient percentage of the aerobic heterotrophs that utilize organic substrate at a high enough rate to impact the maintainable DO concentration during the test, and use a more efficient method of imparting O_2 into the test aquarium so that that the DO can be kept at a high enough concentration to impart less stress on the fish. The arguments against these two approaches are: the technique used to kill off the heterotrophic organisms could conceivably be successful because it also may convert a toxic effluent constituent to a non-toxic end product, and a more efficient way of imparting DO to the aquarium contents, such as the use of pure oxygen instead of air, could also be accused of oxidizing some toxicant.

The removal of sufficient BOD to prevent DO depletion during the toxicity test procedure would be a successful method of eliminating toxicity due to low DO during the test. If biological means are used to achieve such results, there will also be other protection against toxicity test failures, in that biological treatment can also reduce MBAS The only question that must be addressed before serious consideration is given to this alternative is the percentage of influent flow that needs to be treated to reduce toxicity failures to an acceptable frequency. As part of this project, small-scale treatment experiments have been performed to address this question, and they are reported in Section 3.4. Potential options to BOD include: (1) chemically enhanced primary, (2) biological treatment for 25% of ADWF, biological treatment for 50% of ADWF and (4) CEP and biological treatment for 20% of ADWF.

The use of chemically enhanced primary treatment (CEP) is another possibility to reduce the toxicity failures at Iona Island, although its efficacy is not as great as biological treatment for removing BOD, and its ancillary benefits of reducing MBAS or ammonia are expected to be even less. Again, this form of treatment is also being tested at a small-scale level, with results reported in Section 3.4.

3.3.3 <u>Potential Strategies at Lions Gate</u>

The suspected causes of toxicity in the Lions Gate effluent have been relatively well defined and presented in Section 3.2.2 above. The main cause is MBAS and any treatment approach must be focused on that impurity. Partial biological treatment would be very effective in removing MBAS. But as at Iona Island, the question of what fraction of the inflow would have to be treated to achieve the desired results needs to be answered. Toward that end, small-scale treatment tests were conducted at Lions Gate, and those results are reported in Section 3.4 following.

The use of CEP (at least with the addition of flocculating chemicals) can possibly remove some MBAS.

One chemical treatment alternative that can be successful in reducing both MBAS and ammonia concentrations is oxidation using a strong enough oxidant. Chlorine, ozone, and hydrogen peroxide would all fit into this category and are worthy of consideration. Some small-scale treatment testing of this possibility is presented in Section 3.4.

3.3.4 <u>The Role of Demand Management</u>

Demand management alone is unlikely to be able to adequately address the toxicity problems within a reasonable time frame. However it could play a role in facilitating achieving of discharge standards. Once the specific causes of toxicity test failure are better understood a study of the possible impact of demand management could be considered.

3.4 SUMMARY OF SMALL SCALE TESTING

A previous study has identified anionic surfactants, which are measured as methylene blue active substances (MBAS), as the primary cause of toxicity at the Lions Gate WWTP. Sampling and analysis indicates that the influent MBAS concentration at Lions Gate is typically about 2-4 mg/L from 8 AM until late morning, and then increases to a peak as high as 10 to 11 mg/L from about 4 PM to midnight.

At the Iona Island WWTP low dissolved oxygen has been identified as the main cause of fish bioassay failures. The low dissolved oxygen has been attributed to high oxygen demand in the plant effluent samples caused by an active population of viable microbes present in the plant influent, combined with high concentrations of readily-degradable organic material (BOD) in the primary treated effluent.

The pilot-testing program was designed to conduct parallel tests on samples of settled sewage leaving the primary settling tanks. The purpose of the parallel tests was to compare the effectiveness of chemically enhanced primary treatment (CEP) with that of partial biological treatment, and CEP followed by partial biological treatment, in reducing the acute toxicity of the effluent at Lions Gate and Iona Island (acute toxicity as measured by the 96-hour LC_{50} rainbow trout bioassay). At Iona only, an additional batch test was included, to assess the effectiveness of chlorination/dechlorination in improving the chance of passing the 96 hour LC_{50}, by reducing the population of viable bacteria in the plant effluent sample and consequently reducing the initial oxygen demand during the bioassay. Evaluation of partial biological treatment at both plants was undertaken using biological waste sludge taken from the Annacis Island WWTP. Each batch test was done in parallel onsite at either Lions Gate or Iona, using settled sewage from that facility, combined with waste biological sludge from Annacis.

Comparisons among the various treatments should be taken as subjective, that is, since parallel tests were conducted on the same sample of settled sewage each time, relative comparisons regarding the effectiveness of one treatment compared to the others are valid. However, the results should not be projected to full-scale WWTP performance. The results of the acute toxicity bioassay testing at Iona (96 hr LC_{50}) are summarized in Table 3.1.

TABLE 3.1
IIWWTP TOXICITY TEST RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Pass
CEP	Fail	Pass	Pass	Pass	Fail	Pass
25% Biological	Pass	Fail	Fail	Fail	Fail	Pass
50% Biological	Pass	Pass	Pass	Fail	Fail	Pass
CEP+25% Biological	Pass	Pass	Pass	Fail	Fail	Pass
Disinfected	Fail	Fail	Fail	N/A	N/A	N/A

The results of the acute toxicity bioassay testing at Lions Gate (96 hr LC50) are summarized in Table 3.2.

TABLE 3.2
LGWWTP TOXICITY TEST RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Fail
CEP	Fail	Pass	Fail	Pass	Fail	Fail
25% Biological	Fail	Fail	Fail	Fail	Fail	Fail
50% Biological	Fail	Pass	Pass	Fail	Fail	Fail
CEP+25% Biological	Pass	Pass	Pass	Fail	Pass	Pass

Twenty five percent biological treatment was relatively ineffective in improving removal of TSS, TBOD, and SBOD at both Iona Island and Lions Gate compared to the other treatment processes, and was similarly ineffective in reducing the frequency of acute toxicity in the effluent at both plants.

<u>Iona Island WWTP</u>

The samples of primary effluent from the Iona Island WWTP contained material that exerted a high oxygen demand in five of the six batch tests. Oxygen starvation was the most probable cause of the observed 100% lethality within the first hour in the control samples in these five tests. Disinfection of the primary effluent at Iona Island was effective in reducing the initial oxygen demand in the bioassay test, but this did not improve the bioassay results in three of three tests. This indicates that there are additional contributors to acute toxicity besides oxygen demand in the effluent from the Iona WWTP, and that reducing the initial oxygen demand by disinfection will not improve toxicity testing results.

In several cases, the control and treated samples from the Iona Island WWTP contained ammonia concentrations that exceeded the theoretical acute toxicity threshold (according to B.C. Water Quality Guidelines), and this may have contributed to fish mortality.

Chemically enhanced primary (CEP), 50% biological treatment, and CEP followed by 25% biological treatment all showed a 60% improvement in the frequency of toxicity compared to the control. These three processes appear to be approximately equivalent for use as interim improvements at Iona from the standpoint of toxicity reduction and removal of TSS and BOD. None of these processes will produce an effluent that is consistently non-toxic according to the 96 hour LC_{50}, but all can be expected to provide substantial improvements over primary treatment alone.

From the standpoint of effluent quality, chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment should all be considered for interim upgrades at the Iona Island WWTP.

Lions Gate WWTP

The samples of primary effluent from the Lions Gate WWTP contained material that exerted a high oxygen demand in four of the six batch tests. Oxygen starvation was the most probable cause of the observed 100% lethality within the first two hours in the control samples in these four tests.

One hundred percent survival was observed in several treated samples that contained MBAS concentrations in the range 4 mg/L to 6 mg/L, and 90% survival was observed in one sample that contained 7.8 mg/L MBAS. For this limited testing, in three of the six batch tests, samples that were found to be non-acutely toxic according to the bioassay, had higher MBAS concentrations than samples that were acutely toxic within the same batch test.

Based on the test results the following treatments were shown to be effective to a greater or lesser extent in reducing toxicity:

- Chemically enhanced primary treatment,
- 50% biological treatment were similar in effectiveness (33% improvement), and
- CEP + 25% biological treatment was more effective than either of the other processes (83% improvement).

From the standpoint of effluent quality, chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment should all be considered for interim upgrades at the Lions Gate WWTP.

4 FLOWS AND LOADS

4.1 GENERAL

Development of flow and loading projections for the wastewater treatment plants are essential for long-term facility planning. The GVRD's Policy & Planning Department Regional Utility Planning (RUP) Division has established flow and load projections for the Iona Island and Lions Gate Wastewater Treatment Plants up to year 2021. These forecasts represent a current trend scenario and with no change in water use, land use, density, Industrial or C&I waste pretreatment, or infiltration/inflow.

The main objectives of this section of Appendix 3 are:

- To develop a projection methodology, which incorporates the principal drivers of, flow and load.
- To refine the flow and load projections established by RUP by incorporating the best available data and concepts including those of population growth, economic growth, flow and load demand side management.
- To standardize the projection methodology and the analysis of historical data for Iona Island Wastewater Treatment Plant (IIWTP) and Lions Gate Wastewater Treatment Plant (LGWWTP).
- To project flows and loads (BOD, TSS) to the design horizons of interest for Iona island (2021, 2036) and Lions Gate (2031, 2046, 2080).

4.2 POPULATION FORECAST – VSA AND NSSA

The GVRD's Regional Development Division has generated long-range population projections for the region based on 2101 figures developed by BC Stats and the Urban Futures Institute. Up to year 2021, the population is projected to grow as defined in the Livable Region Strategic Plan. The current Growth Management Scenario (GMS) 4.0 for year 2021 forms the basis for the current composite estimate of the regional long-range population scenario to year 2101. The projected regional population total from 2021 to 2101 was straight-lined by decade.

Tables 4.1 and 4.2 present the population projections and ranges for the Vancouver Sewerage Area (VSA) and the North Shore Sewerage Area (NSSA), respectively. Population ranges were developed by the GVRD and reflect the variability due to neighborhood densification, growth capacity limits and growth timing.

TABLE 4.1
VANCOUVER SEWERAGE AREA (VSA) POPULATION PROJECTIONS

Year	Population	Range
2001	616,379	N/a
2006	643,448	N/a
2011	670,518	N/a
2016	697,588	N/a
2021	720,522	700,000 to 750,000
2051	791,000	720,000 to 800,000
2101	990,000	800,000 to 1,100,000

TABLE 4.2
NORTH SHORE SEWERAGE AREA (NSSA) POPULATION PROJECTIONS

Year	Population	Range
2001	173,750	N/a
2006	179,468	N/a
2011	185,187	N/a
2016	190,905	N/a
2021	196,765	200,000 to 220,000
2051	260,000	250,000 to 300,000
2101	370,000	320,000 to 375,000

4.3 FLOW AND LOAD SOURCES

The year 1995 serves as the base year and first year of a comprehensive inventory process undertaken by the GVRD's Policy & Planning – Demand Side Management (DSM) Division. Four primary sources were established as part of the inventory process: Residential, Commercial & Institutional (C&I), Industry, and Trucked Liquid Waste. Detailed inventories of data pertaining to the quantity and quality of wastewater are presented in the 1995 Wastewater Inventory Summary Report and Appendices (GVS&DD, 1995). Table 4.3 summarizes the key parameters for Residential source drawn from the 1995 Wastewater Inventory Report.

TABLE 4.3
PER CAPITA SOURCE CONTRIBUTION

Contribution Sector	Flow, L/c/d		BOD Load, g/c/d		TSS Load, g/c/d	
	Iona	Lions Gate	Iona	Lions Gate	Iona	Lions Gate
Residential	270	270	53	53	61	61

Table 4.4 summarizes the key parameters for C&I source drawn from the updated 1995 to 2002 VSA NSSA Summary. It includes additional data that was not available at the time of the 1995 inventory.

TABLE 4.4
UPDATED C&I SOURCE CONTRIBUTION

Source	Flow, L/c/d		BOD Load, g/c/d		TSS Load, g/c/d	
	Iona	Lions Gate	Iona	Lions Gate	Iona	Lions Gate
C&I	166	55	41	14	21	8

Contributions from the Industry source are developed based on the data collected from the Permitted Industry database. Self-monitoring data is collected and submitted by each regulated industrial discharger as part of the discharge permit requirement. However, it should be noted that not all industries discharging to the GVS&DD system are necessarily permitted; only industries that discharge more than 300 m^3 of non-domestic wastes in 30 days or any “Restricted Wastes” are required to obtain a permit.

There are 117 active permits and 18 active permits in the VSA and the NSSA, respectively, based on the 2002 Permitted Industry database. The top 15 ranked permitted industries in the VSA account for 78%, 93% and 89% of the flow, BOD and TSS contributions respectively from the Permitted Industry source within the sewerage area. The top 3 ranked permitted industries in the NSSA account for 91%, 91% and 91% of the flow, BOD and TSS contributions respectively. Industries not required to obtain a discharge permit are accounted for in a residual category that balances the estimated sector contributions with WWTP influent loadings.

Flow and load contributions from the Trucked Liquid Waste (TLW) can be developed based on haulage records and the two most recent characterization studies of TLW in 1989 and 1997. Currently, the IIWWTP accepts regional TLW generated from both domestic and commercial sources. TLW is not accepted at the LGWWTP.

4.4 PREVIOUS FLOW AND LOAD PROJECTIONS

The GVRD's Policy & Planning – Regional Utility Planning Division has established a standardized methodology for analyzing and projecting flows and loads in order to introduce a level of consistency to the practice of making projections. The resulting flow and load projections for the IIWWTP and the LGWWTP are documented in the Flow and Load Projections 2001 to 2021 report (GVRD, 2001a; GVRD, 2001b). The GVS&DD projection spans a period of 20 years up to 2021, based on the analysis of historic data from 1991 to 1999.

The general procedure for projecting flows and loads consists of determining a per capita unit parameter based on historic data and multiplying that number by the estimated population projection in the sewerage area. The GVRD's Regional Utility Planning Division has analyzed population trends in the VSA and the NSSA for the IIWWTP and the LGWWTP respectively (Section 4.2). To forecast average annual flows and maximum monthly loads, factors determined from an analysis of past flows and loads are applied to the average dry weather flow (ADWF) or the average annual (AA) load.

Table 4.5 summarizes the key projection parameters drawn from the Iona Island Wastewater Treatment Plant - Flow and Load Projections 2001 to 2021 report (GVRD, 2001a).

TABLE4.5
GVRD PARAMETERS FOR THE IIWWTP (BASE YEAR 2000)

Flow Parameter	
Average Dry Weather Flow (ADWF)	704 L/c/d
Factor - Average Annual Flow (AAF)	1.34
BOD Parameter	
Average Annual (AA)	0.125 kg/c/d
Factor - Maximum Monthly Load (MML)	1.31
TSS Parameter	
Average Annual (AA)	0.124 kg/c/d
Factor - Maximum Monthly Load (MML)	1.38

The per capita ADWF for IIWWTP is 704 L/c/d, based on the average unit flows for 1991 and 1993 to 1999 inclusive. The 1992 data, which appears to be significantly lower than other years, is excluded. The per capita average annual (AA) BOD and TSS loads for IIWWTP are 0.125 kg/c/d and 0.124 kg/c/d, based on the maximum of per capita average annual loads between 1991 and 1999 inclusive. The maximum monthly load (MML) is calculated by multiplying the average annual (AA) load by the (MML/AA) factor.

Table 4.6 summarizes the key projection parameters drawn from the Lions Gate Wastewater Treatment Plant - Flow and Load Projections 2001 to 2021 report (GVRD, 2001b).

TABLE 4.6
GVRD PARAMETERS FOR THE LGWWTP (BASE YEAR 2000)

Flow Parameter	
Average Dry Weather Flow (ADWF)	518 L/c/d
Factor - Average Annual Flow (AAF)	1.20
BOD Parameter	
Average Annual (AA)	0.086 kg/c/d
Factor - Maximum Monthly Load (MML)	1.34
TSS Parameter	
Average Annual (AA)	0.104 kg/c/d
Factor - Maximum Monthly Load (MML)	1.43

Similarly, the per capita ADWF for LGWWTP is 518 L/c/d, based on the average unit flows for 1991 to 1999 inclusive. The highest per capita average annual BOD and TSS loads for LGWWTP from 1991 to 1999 inclusive are 0.086 kg/c/d and 0.104 kg/c/d respectively. The maximum monthly load (MML) is calculated by factoring the average annual (AA) load by the (MML/AA) factor.

4.5 DEMAND MANAGEMENT – VSA AND NSSA

4.5.1 Impact of Flow and Demand Management

The GVS&DD flow and load projections were intended to represent a current trend scenario. As such, factors identified as having an impact on flows and loads to the treatment plant were assumed at present. The sole exception is the population of the sewerage area which is projected to grow as defined in the Livable Region Strategic Plan. However, flow and load changes are expected to occur when programs such as combined sewer separation, water conservation, industrial waste management initiatives, reduction in inflow and infiltration, and food waste separation and control are implemented. Therefore, flow and load projections should consider the implementation of the most probable demand management that could potentially reduce the size or timing of future plant upgrade.

4.5.2 Flow Control Measures

4.5.2.1 DSM Water Conservation Program

In order to provide a reasonable indication of the range the possible effects from different drinking water Demand Side Management (DSM) initiatives might have on wastewater flows, the GVRD's DSM Division has developed three scenarios which provide estimates of wastewater flows based on different assumptions about the drinking water DSM initiatives over the next 20 years. These DSM scenarios consider the amount of wastewater flow reduction associated with 'existing water conservation initiatives', 'enhanced water conservation program', and 'aggressive water conservation program' in the VSA and NSSA.

For the purposes of this study, the flow projections included in the "existing water conservation initiatives" scenario are considered as design case estimates. The "enhanced water conservation program' reflects a less likely but more aggressive scenario for the lower boundary of flow projection.

The existing water conservation initiatives include the following:

- Vancouver's requirement that all new and retrofit construction use 6 L toilets;
- The effect of BC Building code (limits the capacity of toilets to at most 13.25 L and places upper bounds on showerhead and faucet flow rates).

On the other hand, an "enhanced water conservation program" includes the following conservation measures:

- Rebates to encourage installation of 6L toilets in all new and retrofit residential construction;
- Home water audits and retrofits; and
- Industrial, Commercial and Institutional (ICI) water audits.

The above water conservation initiatives will only affect the Residential and the Commercial & Institutional (C&I) sector and will reduce the dry weather but not the wet weather flows as shown in Table 4.9.

4.5.2.2 Combined Sewer Overflow (CSO) Program – VSA

As discussed in the GVRD Workshop #1, combined sewer overflows (CSOs) in the VSA will continue for about 50 years. However, some CSO volumetric reductions have occurred as a result of collaborative improvements to the sewer system between the GVRD and member municipalities in the VSA. Notably, the City of Vancouver has implemented a combined sewer separation program that replaced aging combined sewers with separate sanitary and storm sewers. Under the Liquid Waste Management Plan (LWMP), the City of Vancouver is committed to continue with the present combined sewer system separation program at approximately 1 percent of the system per year and the City of Burnaby to implement a similar program, leading to a target elimination of CSOs in the VSA by 2050. However, the continuing combined sewer separation program in the VSA will not affect the peak flows arriving at the IIWWTP during this transition period. The additional capacity resulting from the stormwater separation will be used to intercept combined sewerage that may otherwise have overflowed the sewer system.

4.5.2.3 Infiltration and Inflow (I/I) Reduction Program – NSSA

An Infiltration and Inflow (I/I) reduction program for NSSA is in place to control the groundwater infiltration into the LGWWTP because of the aging sewers in the area. The target for I/I is 11,200 L/ha/d. Municipal sewer infrastructure management programs are targeting a reduction of I/I so that Sanitary Sewerage Overflows (SSO) will be eliminated by 2012.

For the purposes of this study, a 10% reduction of groundwater infiltration, due to sewer repairs, is assumed over the next 20 years starting 2002 for the NSSA.

4.5.2.4 Industry Demand Management

The VSA Permitted Industry Summary shows that industry flow for the VSA has been in decline since 1995. Flow for the NSSA remains roughly the same in 2002 compared to 1995. Some of the reduction in industrial flow is most likely due to the implementation of user sewer fees instituted in 1998. It is believed that the sewer fees will continue in the future.

4.5.3 Load Control Measures

4.5.3.1 Food Waste Discharges

The impact of food wastes (FW) discharged to the sewer system through garburator use was documented in a 1998 report “Food Waste Discharges to the Sewer System” prepared by Compass Resource Management. Each kilogram of food waste (measured as wet solids) was estimated to contribute about 0.2 kg of BOD and 0.3 kg of TSS (measured as dry solids) load to the treatment plant. The GVRD’s

DSM Division has initiated a study to determine whether policy provisions should be developed to discourage or eliminate food grinder use. The initiatives could affect both the Residential and the C&I sources. Table 4.7 and 4.8 present the estimated food waste load at IIWWTP and LGWWTP in 2002 based on values prepared by the DSM.

TABLE 4.7
ESTIMATED GARBURATOR LOAD (2002) FOR IIWWTP

	BOD Load, t/yr (= 0.2 x FW)	TSS Load, t/yr (= 0.3 x FW)	Food Waste (FW) to Sewer, t/yr
Residential	665	997	3,324
C&I	340	510	1,700

Note: Food Waste (FW)

TABLE 4.8
ESTIMATED GARBURATOR LOAD (2002) FOR LGWWTP

	BOD Load, t/yr (= 0.2 x FW)	TSS Load, t/yr (= 0.3 x FW)	Food Waste (FW) to Sewer, t/yr
Residential	188	283	942
C&I	67	101	335

Note: Food Waste (FW)

Garburators/food grinders are used in both the residential household and the C&I sources. A recent Angus Reid survey, commissioned by the GVRD, estimated that one third of households had food grinders in 2000. It is assumed that 35% of the food waste generated in households is eligible for food grinder disposal, and is disposed in this manner by households equipped with food grinders. C&I sources that may use food grinders include food service, food-retail, schools and hospitals. It is assumed that 20% of businesses and institutions are equipped with food grinders (except food-retail, where 5% is assumed), and that 30% of the food waste generated by C&I sources with food grinders is discharged to the sewer.

In the absence of any food waste control initiatives, it is assumed that 80% of new households would be equipped with food grinders. For the purpose of this study, a design case assumes the same. The lower boundary of load projections includes a more aggressive DSM scenario where it is assumed that food grinders in the residential households are reduced from one third of all households in 2002 to 10% of all households in the design year.

4.5.3.2 Industry Demand Management

The potential DSM reduction scenario assumes that industry loadings can be reduced by 25% in 2021 relative to the base case in 2002. The VSA and NSSA Permitted Industry Summary indicates that industry loadings have declined significantly since 1995. Some of this decline is likely due to the introduction of industry sewer user fees in 1998, phased in over a 4-year period. The decline may also be related to the general economic climate, competitive factors, and company specific issues. It is believed that the sewer user fees will remain and will continue to play a role in reducing loads from existing businesses. Pollution prevention and sustainable region initiatives may also contribute to further reductions. Nevertheless, there is a need to be cautious in projecting reductions in industry loadings, considering that existing businesses may be approaching the point of diminishing returns. Moreover, part of this load reduction from existing businesses could be offset by demand from new businesses. In the NSSA, due to a small industrial base, the addition of a single major player or a step change from one contributor can make a significant difference in the overall industry load contribution. In 2001, the industry TSS loading increased by 33% with the addition of a new and relatively large industrial discharger.

For the purposes of this study, a less aggressive potential DSM reduction scenario is assumed. New business loading is assumed to grow at the lower boundary of the population growth rate as given in Growth Management Scenario 4.0. Current industry loading is assumed to grow at a reduced 50% of the population growth rate, reflecting improvements in existing business practices.

4.5.4 Potential Impacts of Flow and Load Controls

4.5.4.1 Flow - DSM Water Conservation Program

Table 4.9 shows the effects of GVRD’s existing and enhanced water conservation initiatives on the residential and C&I flow projections for IIWWTP and LGWWTP.

TABLE 4.9
EFFECTS OF DSM’S WATER CONSERVATION INITIATIVES ON AVERAGE DRY WEATHER FLOW

Source	Enhanced (L/c/d)		Existing (L/c/d)	
	2002	2021	2002	2021
Iona Island WWTP				
Residential	264	214	264	220
C&I	166	153	166	166
Lions Gate WWTP				
Residential	270	232	270	243
C&I	55	51	55	55

The above estimates are developed using assumptions about the relationship between population and water use, the likely effect of water conservation initiatives on water use, and the relationship between water use and wastewater generation. Estimates of water use in the C&I sector are based on data in the report Regional Water Demand by Sector, GVRD, 2001. 80% of the water used in the C&I sector is assumed to enter the wastewater system. Implementation of the water conservation initiatives included in the DSM scenario is assumed to begin in 2004.

4.5.4.2 Flow – CSO and I/I Reduction

Table 4.10 shows the effects of groundwater infiltration contribution to IIWWTP and LGWWTP with the repair of sewers in both VSA and NSSA from 2002 in a 20-year period to 2022:

TABLE 4.10
EFFECTS OF SEWER REPAIR ON GROUNDWATER INFILTRATION DURING AVERAGE DRY WEATHER FLOW

Source	Iona Island (ML/d)		Lions Gate (ML/d)	
	2002	**2022**	**2002**	**2022**
Groundwater Infiltration	140	126	28	25

4.5.4.3 Flow – Industry Demand Management

The potential DSM reductions may be enhanced by an expected shift in industry trends. For the purposes of this study, new businesses are assumed to grow at the lower boundary of population growth rate as given in Growth Management Scenario 4.0 and existing businesses to grow at 50% of population growth. Table 4.11 shows the assumed annual growth rate of industrial flow from both new and existing businesses in the VSA up to year 2036 and in the NSSA up to year 2046.

TABLE 4.11
ANNUAL GROWTH RATE OF INDUSTRIAL FLOW FROM NEW AND EXISTING BUSINESSES IN THE VSA AND THE NSSA

Year	Iona Island (VSA)		Lions Gate (NSSA)	
	New Businesses	Existing Businesses	New Businesses	Existing Businesses
2002 to 2021	0.63%	0.31%	0.71%	0.35%
Beyond 2021	0.09%	0.05%	0.75%	0.38%

The net result of these factors is that industrial flow will change only marginally. These growth rates were applied to the current industrial flow of 25 ML/day.

4.5.4.4 Load – Food Waste Discharges

Based on the same assumptions used in the 1998 report “Food Waste Discharges to the Sewer System” (Compass, 1998) to estimate the food waste (FW) loadings and related BOD and TSS loads, the per capita BOD and TSS contributions from food waste loadings are 0.01 kg/c/d and 0.02 kg/c/d respectively, assuming 33% of the total households are equipped with garburators. The maximum target of the DSM food waste discharges reduction scenario assumes that households equipped with garburators will reduce from one-third of total households to 10% by 2036 for IIWWTP and by 2046 for LGWWTP. The food waste discharges to sewer from C&I sources are assumed to reduce to 10% of the C&I per capita food waste loadings in 2002 by year 2036 for IIWWTP and by year 2046 for LGWWTP.

Table 4.12 and Table 4.13 show the resulting maximum monthly BOD and TSS contributions from the Residential and C&I sources to IIWWTP and LGWWTP with the potential DSM food waste discharges reductions implemented.

TABLE 4.12
BOD CONTRIBUTION (MAX. MONTH) WITH REDUCTIONS IN FOOD WASTE DISCHARGES TO SEWER

Source	Iona (kg/c/d)		Lions Gate (kg/c/d)	
	2002[c]	2036	2002[c]	2046
Residential	0.070	0.066[a] + 0.001[b]	0.071	0.067[a] + 0.001[b]
C&I	0.050	0.048	0.020	0.019

Notes:
[a] BOD contribution from Residential sources with complete elimination of food waste discharges to sewer
[b] 10% of total households contribute 0.001 kg/c/d BOD loads by food waste discharges to sewer
[c] Based on average annual source contribution (Table 4.3 and Table 4.4) multiplied by BOD max. monthly factor (Table 4.5 and 4.6)

TABLE 4.13
TSS CONTRIBUTION (MAX. MONTH) WITH REDUCTIONS IN FOOD WASTE DISCHARGES TO SEWER

Source	Iona (kg/c/d)		Lions Gate (kg/c/d)	
	2002[c]	2036	2002[c]	2046
Residential	0.080	0.078[a] + 0.002[b]	0.090	0.081[a] + 0.002[b]
C&I	0.030	0.027	0.010	0.008

Notes:
[a] TSS contribution from Residential sources with complete elimination of food waste discharges to sewer
[b] 10% of total households contribute 0.002 kg/c/d TSS loads by food waste discharges to sewer
[c] Based on average annual source contribution (Table 4.3 and Table 4.4) multiplied by TSS max. Monthly factor (Table 4.5 and 4.6)

4.5.4.5 Load – Industry

The current DSM mandate is not to manage industrial growth, but to promote sustainable practices within businesses. Table 4.14 and 4.15 show the actual maximum monthly records for BOD and TSS from industrial source in the 2002 VSA and NSSA summary inventories as compared to the baseline maximum monthly loads derived from historic factors. The baseline numbers are calculated by multiplying the BOD and TSS contributed by industry in the "VSA and NSSA Sampling – Permitted Industry" in 2002, by the corresponding maximum month factors (Section 4.4).

For the purposes of this study, the lesser of the two values is assumed to be the load parameter to reflect a potential scenario where there is a shift to industries that do not generate high wastewater loads.

TABLE 4.14
COMPARISON BETWEEN ACTUAL BOD RECORDS AND BASELINE BOD LOADS (MAX. MONTH)

Source	Iona (t/d)		Lions Gate (t/d)	
	2002	Baseline	2002	Baseline
Industrial	21.3 (May)	23.6	1.11 (August)	0.54

TABLE 4.15
COMPARISON BETWEEN ACTUAL TSS RECORDS AND BASELINE TSS LOADS (MAX. MONTH)

Source	Iona (t/d)		Lions Gate (t/d)	
	2002	Baseline	2002	Baseline
Industrial	7.1 (May)	6.8	1.08 (April)	0.858

4.6 IONA ISLAND WWTP

4.6.1 General

Located on Iona Island, the IIWWTP serves most of the City of Vancouver, the University Endowment Lands, and parts of Burnaby and Richmond. Current permitted flow capacity at the plant is 1530 ML/d. The catchment area is mostly developed with extensive residential, commercial and industrial zones. The IIWWTP receives flow from the Vancouver Sewerage Area (VSA), primarily from combined sewers, which intercept stormwater runoff in addition to sanitary flow. Weather patterns in Greater Vancouver result in increased flows to the plant during winter months when there is the greatest amount of precipitation. Wastewater sources in the VSA are primarily residential but do include a significant component of commercial/institutional and industrial flows. In the past, most permit exceedances have occurred at the IIWWTP during low flow periods in the summer when infiltration/inflow is low, resulting in higher strength flows.

4.6.2 Evaluation of Historic Flow Data

Average dry weather flow (ADWF) is defined as the 25th percentile flow of average daily flows over the calendar year. Based on a review of historic data, it is reasonable to use the average dry weather flow, which corresponds to flow with higher contaminant concentration. For the purposes of this study, ADWF includes contributions from the residential, industrial and commercial/institutional sources and groundwater infiltration. The ADWF will be used to establish the design criteria for the interim facility upgrade.

Residential, Commercial & Institutional (C&I), Industrial and Trucked Liquid Waste represent the primary wastewater contribution sources. However, stormwater and groundwater seepage from surrounding creeks, and plant recycle streams also contribute to the total plant influent. For the purposes of this study, “Groundwater Infiltration” includes inflow and all flows that are not accounted for by the primary sources during dry weather flow periods.

The Regional Utility Planning Division of the GVRD has established a per capita ADWF of 704 L/c/d for IIWWTP based on historic average unit flows from 1991 to 1999. Table 4.16 summarizes the relative flow contribution of each source within the VSA to the Iona Island WWTP in year 2002 based on the total per capita ADWF of 704 L/c/d.

TABLE 4.16
FLOW DISTRIBUTION BASED ON ADWF (704 L/c/d)

Source	**Distribution (2002)**	**Flow, L/c/d**	**Flow, ML/d**
Residential	38%	270	167
C&I	24%	166	103
Industry	6%	41	25
Groundwater Infiltration	32%	227	140
Total	100%	704	435

Residential flow is prorated from the 1995 Wastewater Inventory flow parameter of 270 L/c/d, based on a service population of 617,658 in year 2002. C&I sector flow is prorated based on the updated per capita contribution of 166 L/c/d (Table 4.16) and a service population of 617,658. Industrial wastewater flow is obtained from the Permitted Industry database (117 active permits in 2002) prepared by the GVRD or the purpose of this study, industries not required to obtain a discharge permit are considered to have negligible discharges. Flow contribution from TLW comprised less than 1% of the total flow and is therefore deemed insignificant.

4.6.3 Evaluation of Historic Load Data

The GVS&DD 1995 Wastewater Inventory established a baseline profile for the characteristics of the wastewater received by the IIWWTP. The Residential source was identified as the leading contributor of BOD and TSS. The Industrial source ranked second in BOD and TSS contributions. However, substantial industrial loading reductions have occurred in the 7 years since the first wastewater inventory was compiled in 1995. This reduction was possibly influenced by the introduction of sewer use fees beginning in 1998. In the VSA, Permitted Industry BOD and TSS loadings declined 50% and 65% respectively since 1995. The average industrial BOD and TSS contributions in 2002 were 18.0 t/d and 4.9 t/d respectively.

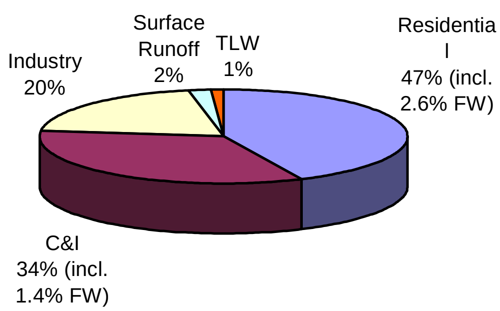
Table 4.17 presents the key characteristics of the wastewater received by the Iona Island WWTP historically. Parameters are drawn from the Iona Island WWTP – Flow and Load Projections 2001 to 2021 prepared by the GVRD (GVRD, 2001a).

TABLE 4.17
IIWWTP HISTORIC WASTEWATER CHARACTERISTICS

All sources	BOD, g/c/d	TSS, g/c/d	Data Source
Average Annual (AA)	113	108	Based on average historic data from 1991 to 1999
Max. AA	125	124	BOD: Based on 1995 data TSS: Based on 1994 data
Min. AA	90	92	Based on historic data of 1992

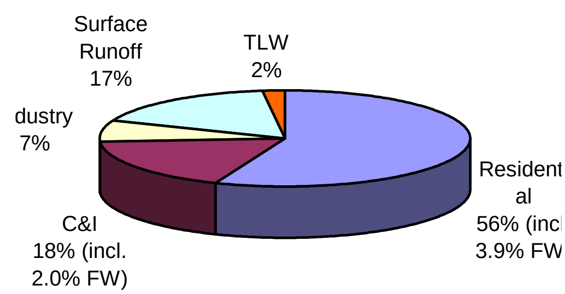
Figures 4.1 and 4.2 show the relative BOD and TSS loads from each source within the VSA to the Iona Island WWTP in year 2001/2002 based on the total average annual (AA) loading at the plant.

FIGURE 4.1
IIWWTP BOD LOADING (2001)



Total BOD (AA) = 74.5 t/d

FIGURE 4.2
IIWWTP TSS LOADING (2002)



Total TSS (AA) = 70.0 t/d

Note: Food Waste (FW)

Loads from the Residential and C&I sources are prorated from the BOD and TSS parameters presented in the following Table 4.18, based on a service population of 612,244 in year 2001 and a service population of 617,658 in year 2002. Industrial load is obtained from the Permitted Industry database prepared by the GVRD's Policy & Planning Department – DSM Division. Loadings arising from garburator use are included in Residential and C&I source contributions.

TABLE 4.18
RESIDENTIAL AND C&I SOURCE CHARACTERISTICS

Source	BOD, g/c/d	TSS, g/c/d	Data Source
Residential	53	61	Based on the GVS&DD 1995 Wastewater Inventory Report (GVS&DD, 1995)
C&I	41	21	Based on the updated VSA Summary

It should be noted that the wastewater characterization for the Residential and C&I sources shown in Table 4.18 above is an estimate based on limited sampling data developed in 1995 and literature values where available. Nevertheless, when applied to the total loadings at the Iona Island WWTP, these parameters produce a distribution profile that is considered reasonable among the primary contribution sources.

The “TLW” category in Figures 4.1 and 4.2 includes domestic TLW but not commercial TLW. Currently, the Iona Island WWTP is operated to accept domestic and commercial TLW in two separate receiving systems. The domestic TLW is discharged directly by gravity into the plant influent siphons that enters the main treatment process. The commercial TLW is pre-treated by a stand-alone facility before being discharged into the north influent channel of the main plant. Influent samples are collected at the influent pumps; therefore, only domestic trucked liquid waste discharged to the plant influent siphons are inventoried as part of the total plant loading.

The VSA is comprised primarily of combined sewers which intercept stormwater runoff in addition to sanitary flow. The average annual (AA) load, defined as the BOD or TSS load averaged over a calendar year, is also a function of the amount of stormwater that enters the system during wet weather months. On wet weather days, inflow to the sewer system has a tendency to reduce the concentration of BOD and TSS through dilution; however, the total load to the plant may increase. For the purposes of this study, “Surface Runoff” includes all loads that are not accounted for by the primary contribution sectors on an average annual basis.

While BOD contribution due to surface runoff appears to be significantly higher in 2001 than other years, it is estimated to be less than 2% of the total BOD plant load in 2001. On the other hand, surface runoff TSS contribution in 2002 is estimated to be approximately 17% of the total plant load.

4.6.4 Base Case Projection for Facility Planning

Under the Liquid Waste Management Plan (LWMP), full secondary treatment upgrade at Iona Island WWTP is required by 2020. For the purposes of this study, flow and loads projected to the year 2036 will be used to design the secondary treatment facility at IIWWTP.

The basic theory used in this study to project flow and loads for facility upgrade at the Iona Island WWTP and the Lions Gate WWTP is the same standardized methodology used by the GVRD in the Flow and Load Projections 2001 to 2021 report (GVRD, 2001a, GVRD, 2001b). The base case projection developed in this study for the IIWWTP spans a projection period of 35 years up to year 2036. The population trend in the VSA developed by the GVRD for the IIWWTP (Section 4.2, Table 4.1) is used in this study to estimate the projection growth rate. Flow is projected on an average dry weather flow (ADWF) basis. Contaminant loads are projected on a maximum monthly (MM) basis by multiplying a peak factor (Section 4.4) to the average annual (AA) load projection. Flow and loading projections are further categorized by contribution sectors based on the analysis of current trends and recent distribution profile.

Table 4.19 summarizes the base case parameters used in this study to project flow received by the Iona Island WWTP.

TABLE 4.19
IIWWTP AVERAGE DRY WEATHER FLOW (ADWF) PROJECTION

Flow Source	Base Case	Growth Rate
Groundwater Infiltration	140 ML/d	In accordance with population growth in Table 4.2 (720,522 by 2021 and 755,000 by 2036)
Residential	270 Lcpd	
C & I	166 Lcpd	
Industry	25 ML/d	

Table 4.20 summarizes the base case parameters used in this study to project BOD loads received by the Iona Island WWTP on a maximum monthly basis.

TABLE 4.20
IIWWTP MAXIMUM MONTHLY (MM) BOD PROJECTION

BOD Source	Base Case		Growth Rate
	Per capita AA	MM = 1.34 x AA	
Residential	53 g/c/d	0.07 kg/c/d	In accordance with population growth in Table 4.2 (720,522 by 2021 and 755,000 by 2036)
C & I	41 g/c/d	0.05 kg/c/d	
Industry	29 g/c/d	0.04 kg/c/d	
Trucked Liquid Waste (Domestic – raw loading)	-	0.5 t/d	
Trucked Liquid Waste (Commercial – raw loading)	-	1.6 t/d	
Surface Runoff	2 g/c/d	1.8 t/d	To remain constant percentage of AA

Table 4.21 summarizes the base case parameters used in this study to project TSS loads received by the Iona Island WWTP on a maximum monthly basis.

TABLE 4.21
IIWWTP MAXIMUM MONTHLY (MM) TSS PROJECTION

TSS Source	Base Case		Growth Rate
	Per capita AA	MM = 1.38 x AA	
Residential	61 g/c/d	0.08 kg/c/d	In accordance with population growth in Table 4.2 (720,522 by 2021 and 755,000 by 2036)
C & I	21 g/c/d	0.03 kg/c/d	
Industry	8 g/c/d	0.01 kg/c/d	
Trucked Liquid Waste (Domestic – raw loading)	-	1.9 t/d	
Trucked Liquid Waste (Commercial – raw loading)	-	4.0 t/d	
Surface Runoff	18 g/c/d	15 t/d	To remain constant percentage of AA

4.7 LIONS GATE WWTP

4.7.1 General

Located west of the First Narrows crossing on the, North Shore. The Lions Gate WWTP receives sanitary flow from the North Shore Sewerage Area (NSSA) which comprises the City of North Vancouver, the District of West Vancouver and the District of North Vancouver. The service population of NSSA in 2002 was 175,036. The catchment comprises primarily residential areas, but also includes commercial/institutional (C&I) and industrial area. Current permitted dry weather flow to the plant is 102 ML/d. During the wet winter months, flows to the plant increase substantially. Precipitation seeps through the ground and infiltrates into the older sewers in the NSSA. This, together with the inflow, becomes a significant portion of the total flow to the LGWWTP during wet weather. After primary treatment, the effluent is disinfected and discharged to Burrard Inlet via an outfall.

4.7.2 Evaluation of Historic Flow Data

Average dry weather flow (ADWF) is defined as the 25th percentile flow of average daily flows over the calendar year. Based on a review of historic flow data for LGWWTP, it is reasonable to use the ADWF for LGWWTP flow projection. For the purposes of this study, ADWF includes contributions from the residential, industrial and C&I sectors and base infiltration. The ADWF will be used to establish a base case scenario for the design criteria for the interim facility upgrades.

The primary wastewater sources for the LGWWTP are Residential, C&I, and Industrial sectors. No Truck Liquid Waste is allowed in LGWWTP. However, infiltration/inflow including groundwater seepage and stormwater inflow also contribute to the total plant influent. For the purposes of this study, the term “Groundwater Infiltration” includes inflow and all flows that are not accounted for by the primary sources during dry weather flow periods.

The Regional Utility Planning Division of the GVRD has established a per capita ADWF of 518 L/c/d for LGWWTP based on historic average unit flows from 1991 to 1999. Table 4.22 summarizes the relative flow contribution of each sector within the NSSA to the Lions Gate WWTP in the year 2002 based on the historic average per capita ADWF of 518 L/c/d.

TABLE 4.22
LGWWTP FLOW DISTRIBUTION BASED ON ADWF (518 L/c/d)

Source	Distribution (2002)	Flow, ML/d (2002)
Residential	52%	47
C&I	11%	9.7
Industry	7%	6.0
Groundwater Infiltration	30%	27
Total	100%	90

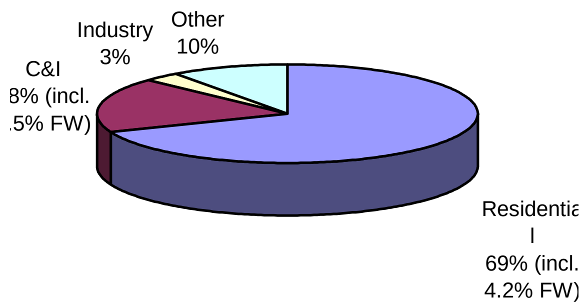
Flows from the Residential and C&I sectors are prorated from the 1995 Wastewater Inventory's flow parameters of 270 L/c/d and 55 L/c/d (Table 4.4), respectively, based on a service population of 175,036 in year 2002. Industrial wastewater flow is obtained from the NSSA Permitted Industry database (18 active permits in 2002) prepared by the GVRD's Policy & Planning Department. For the purpose of this study, industries not required to obtain a discharge permit are considered to have negligible discharges.

4.7.3 Evaluation of Historic Load Data

The GVS&DD 1995 Wastewater Inventory established a baseline profile for the characteristics of the wastewater received by the LGWWTP. The Residential sector was identified as the leading contributor of BOD and TSS according to the 1995 Wastewater Inventory. The second leading contributor of BOD and TSS in the system was predominantly inflow/infiltration.

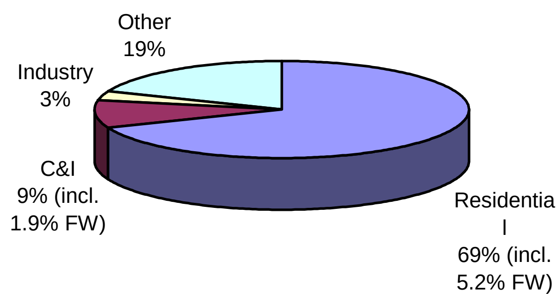
Figures 4.3 and 4.4 show the relative BOD and TSS load from each source within the NSSA to the Lions Gate WWTP in year 2002.

FIGURE 4.3
LGWWTP BOD LOADING (2002)



Total BOD (AA) = 12.3 t/d

FIGURE 4.4
LGWWTP TSS LOADING (2002)



Total TSS (AA) = 14.8 t/d

Note: Food Waste (FW)

Based on an analysis of historic data from 1991 to 1999 (GVRD, 2001b), the Regional Utility Planning Division of the GVRD established the average annual (AA), maximum AA and minimum AA for BOD and TSS contribution to the LGWWTP. These values are summarized in Table 4.23.

TABLE 4.23
LGWWTP HISTORIC WASTEWATER CHARACTERISTICS

	BOD, g/c/d	**TSS, g/c/d**	**Data Source**
Average Annual (AA)	77	88	Based on average historical data from 1991 to 1999
Maximum AA	86	104	BOD: Based on 1996 data TSS: Based on 1998 data
Minimum AA	61	68	Based on 1991 data

Table 4.24 presents the key characteristics of the wastewater received by LGWWTP. Parameters are drawn from the GVS&DD 1995 Wastewater Inventory Report (GVS&DD, 1995) and the Lions Gate WWTP – Flow and Load Projections 2001 to 2021(GVRD, 2001b) prepared by the GVRD. Loads from the Residential and C&I sectors are prorated from the 1995 Wastewater Inventory's BOD and TSS parameters based on a service population of 175,036 in year 2002. Industrial loading is obtained from the Permitted Industry database (18 active permits in 2002) prepared by the GVRD. Loadings arising from garburator use are included in Residential and C&I sector contributions.

TABLE 4.24
RESIDENTIAL AND C&I SOURCE CHARACTERISTICS

Source	BOD (AA), g/c/d	TSS (AA), g/c/d	Data Source
Residential	53	61	Based on GVS&DD 1995 Wastewater inventory
C&I	19	28	Based on updated NSSA Summary

It should be noted that the wastewater characterization for the Residential and C&I sources shown in Table 4.24 is an estimate based on limited sampling data developed in 1995 and literatures values where available. Nevertheless, when applied to the total loadings at the LGWWTP, these parameters produce a distribution profile that is considered reasonable among the primary contribution sources.

4.7.4 Base Case Projection for Facility Planning

Under the Liquid Waste Management Plan (LWMP), full secondary treatment upgrade at Lions Gate is required by 2030. For the purposes of this study, flow and load projected to the year 2046 will be used to design the secondary treatment facility at LGWWTP. In order to provide for diversion of flow to the IIWWTP from the North Shore, flow and load projected to 2081 will be used to size new sewers.

The basic theory used in this study to project flow and load for facility upgrade at LGWWTP is the same as the standardized methodology used by the GVRD in the Flow and Load Projections 2001 to 2021 report (GVRD, 2001b). The base case projection developed in this study for the LGWWTP spans a period of 80 years from 2001 to 2081. The population trend in the NSSA developed by the GVRD for LGWWTP (Section 4.2, Table 4.2) is used in this study to estimate the projection growth rate (Table 4.25).

TABLE 4.25
NORTH SHORE SEWERAGE AREA (NSSA) POPULATION GROWTH RATE

Year	Population	Growth rate per annum (%)
2001	173,750	-
2006	179,468	0.65
2011	185,187	0.63
2016	190,905	0.61
2021	196,765	0.61
2051	260,000	0.93
2101	370,000	0.71

Flow is projected based on an average dry weather flow (ADWF). Loads are projected on a maximum month (MM) basis by factoring the average annual (AA) load projection by the factors presented in Table 4.6. Flow and load projections are further categorized by contributing sectors based on the analysis of current trends and recent distribution profile.

Table 4.26 summarizes the base case parameters used in this study to project flow received by the Lions Gate WWTP.

TABLE 4.26
LGWWTP AVERAGE DRY WEATHER FLOW (ADWF) PROJECTION

Flow Source	Base Case	Growth Rate
Groundwater Infiltration	28 ML/d	In accordance with population growth in Table 4.2 (196,765 by 2021 and 248,000 by 2046)
Residential	270 L/c/d	
C & I	55 L/c/d	
Industry	6 ML/d	

Table 4.27 summarizes the base case parameters used in this study to project BOD loads received by the Lions Gate WWTP on a maximum monthly basis.

TABLE 4.27
LGWWTP MAXIMUM MONTH (MM) BOD PROJECTIONS

BOD Source	Base Case		Growth Rate
	Per capita AA	MM = 1.34 x AA	
Residential	53 g/c/d	71 g/c/d	In accordance with population growth in Table 4.2 (196,765 by 2021 and 248,000 by 2046)
C & I	14 g/c/d	19 g/c/d	
Industry	2 g/c/d	3 g/c/d	
Surface Runoff	8 g/c/d	1.8 t/d	To remain constant percentage of AA

Table 4.28 summarizes the base case parameters used in this study to project TSS loads received by the Lions Gate WWTP on a maximum monthly basis.

TABLE 4.28
LGWWTP MAXIMUM MONTH (MM) TSS PROJECTIONS

TSS Source	Base Case		Growth Rate
	Per capita AA	MM = 1.43 x AA	
Residential	61 g/c/d	87 g/c/d	In accordance with population growth in Table 4.2 (196,765 by 2021 and 248,000 by 2046)
C & I	8 g/c/d	11 g/c/d	
Industry	3 g/c/d	4 g/c/d	
Surface Runoff	16 g/c/d	4.1 t/d	To remain constant percentage of AA

4.8 PROJECTION ENVELOPE FOR FACILITY PLANNING

The following sections present summary tables and graphs to depict the impact of future conditions and various Demand Side Management (DSM) scenarios on flow and load projections for the Iona Island WWTP and the Lions Gate WWTP. The approach used in this study to project the lower and upper envelope for flows and loads is the same standardized methodology used for the base case projection (Sections 4.6.4 and 4.7.4). In addition, a design case is developed to reflect less aggressive demand side management programs combined with high growth rate.

Flow and load parameters developed in this study for the lower and upper projection envelopes are based on the following:

(1) Separate loads and flow projections were prepared for the following four contributors of wastewater: residential, commercial and institutional, industrial and groundwater infiltration.

(2) Various DSM scenarios proposed by GVRD (Section 4.5).

(3) Population projections are estimated in accordance with the population ranges developed by the GVRD's Regional Development Division for the VSA and the NSSA (Section 4.2).

(4a) The upper and lower envelopes for the flows and loads for Iona Island WWTP were prepared by adding the lower and upper envelopes for the Trucked Liquid Waste and for the four contributors in (1) above.

(4b) The upper and lower envelopes for the flows and loads for Lions Gate WWTP were prepared by adding the lower and upper envelopes for the contributors in (1) above.

Flow and load parameters developed in this study for the design case projection are based on the following:

(1) Flow projection from groundwater infiltration source is estimated as lower envelope plus 80% of the difference between upper and lower envelopes.

(2) Separate loads and flow projections for residential, commercial and institutional contributors were prepared based on various potential flow and load control measures under the design case discussed in Sections 4.5.2 and 4.5.3.

(3) Population projection is estimated as lower boundary plus 80% of the difference between the upper and lower boundaries.

(4a) The design case projections for the flows and loads for Iona Island WWTP were prepared by adding the individual contributions from residential, commercial and institutional, industrial, Trucked Liquid Waste and groundwater infiltration sources under the design case.

(4b) The design case projections for the flows and loads for Lions Gate WWTP were prepared by adding the individual contributions from residential, commercial and institutional, industrial and groundwater infiltration sources under the design case.

4.8.1 Iona Island WWTP

Tables 4.29a, 4.30a and 4.31a summarize previous findings based on the evaluation of historic flow and load data (Sections 4.6.2 and 4.6.3) and present the key projection parameters for lower and upper envelopes in comparison to the base case parameters for flow and load projections (Section 4.6.4). Tables 4.29b, 4.30b and 4.31b show the design cases for Average Dry Weather Flow (ADWF), BOD (Max. Month) and TSS (Max. Month) where key projection parameters were adjusted to reflect the most probable impact of demand side management. Spreadsheets were developed based on the design criteria and growth rates summarized in Tables 4.29, 4.30, and 4.31. Graphical representations are shown in Figures 4.5 to 4.12.

4.8.1.1 Flow - ADWF

Key parameters for lower and upper envelopes

Table 4.29a summarizes the factors affecting the lower and upper envelopes of ADWF projections up to year 2036 for IIWWTP. For the purposes of this study, flow projected to the year 2036 will be used to design the secondary treatment facility at IIWWTP.

Upper and Lower Projection Envelopes

Figure 4.5 shows the graphical representation of individual ADWF lower and upper projection envelopes categorized by the primary contribution sources (Residential, Commercial & Institutional, Industry and Groundwater Infiltration) to IIWWTP from 2002 to 2036.

Figure 4.6a compares the upper and lower projection envelopes developed in this study based on the ADWF parameters shown in Table 4.19 to previous projections documented in the Flow and Load Projections 2001 to 2021 report (GVRD, 2001a) For the purpose of this study, the GVRD projection based on a per capita ADWF of 704 L/c/d is extended to year 2036. The lower and upper boundaries of the GVRD projection envelope are based on the lower and upper ranges of historic per capita ADWF (636 L/c/d and 750 L/c/d respectively) documented in the same report. It is worth noting that the proposed ADWF upper envelope developed in this study is very similar to the GVRD baseline projection.

Design Case

Table 4.29b shows the adjusted parameters for ADWF projection under the design case based on the potential impact of the following DSM scenario:

“Existing water conservation initiatives” scenario (Section 4.5.2.1).

Figure 4.6b compares the design case projection in relation to the upper and lower projection envelopes. The design case approaches a point of diminishing returns in year 2021 at 460ML/d.

4.8.1.2 Load – BOD and TSS

Key parameters for lower and upper projections

Tables 4.30a and 4.31a summarize the factors affecting the lower and upper envelopes of BOD (Max. Month) and TSS (Max. Month) projections, respectively, up to year 2036 for IIWWTP. For the purposes of this study, maximum monthly (MM) loads projected to the year 2036 will be used to design the secondary treatment facility at IIWWTP. The MM load is calculated by multiplying the average annual (AA) load by the MML/AA factor (Section 4.4). The peak factors for BOD and TSS at IIWWTP are 1.31 and 1.38 respectively.

As discussed in Section 4.5.4.4, the per capita BOD and TSS contributions from food waste loadings are 0.01 kg/c/d and 0.02 kg/c/d respectively. With complete elimination of food waste discharges to sewer, the per capita BOD (MM) and TSS (MM) contributions from the Residential source are estimated to be 0.066 kg/c/d and 0.067 kg/c/d respectively. The lower envelope of the BOD (MM) and TSS (MM) projections from the Residential source consists of a universal contribution (0.066 kg/c/d for BOD, 0.067 kg/c/d for TSS) and an additional per capita contribution due to food waste loadings (0.01 kg/c/d for BOD, 0.02 kg/c/d for TSS) based on households that have food grinders (to be reduced from one-third of total households to 10% by 2036). In the absence of any food waste discharge control initiatives, the proportion of residential households equipped with food grinders is likely to increase in the future years. The upper envelope of the BOD (MM) and TSS (MM) projections from the Residential source consists of a per capita base case contribution (0.070 kg/c/d for BOD, 0.080 kg/c/d for TSS) which includes food waste loadings from the existing households and an additional per capita contribution due to food waste loadings (0.01 kg/c/d for BOD, 0.02 kg/c/d for TSS) based on 80% of the new households.

Upper and Lower Projection Envelopes

Figure 4.7 shows the graphical representation of individual BOD (MM) lower and upper projection envelopes categorized by Residential, Commercial & Institutional, and Industry sources to IIWWTP from 2002 to 2036. As discussed in Section 4.5.4.5, the lesser value between the actual BOD maximum monthly record and the baseline maximum monthly BOD derived from historic factor is used for lower envelope projection. It is worth noting that, for IIWWTP, the upper envelope is developed based on the Maximum Month Loading (MML) record of BOD in 2002 and the lower envelope is developed by factoring the 2002 AA loading of BOD by 1.31.

Figure 4.8 and 4.9a compare the upper and lower projection envelopes developed in this study based on the maximum monthly BOD parameters shown in Table 4.30a to previous BOD projection documented in the Flow and Load Projections 2001 to 2021 report (with and without TLW). The GVRD projection was developed by factoring the upper boundary of the historic AA per capita BOD load (0.125 kg/c/d).

Figure 4.10 shows the graphical representation of individual TSS (MM) lower and upper projection envelopes categorized by Residential, Commercial & Institutional, and Industry sources to IIWWTP from 2002 to 2036. As discussed in Section 4.5.4.5, the lesser value between the actual TSS maximum monthly record and the baseline maximum monthly TSS derived from historic factor is used for lower envelope projection. It is worth noting that, for IIWWTP, the lower envelope is developed based on Maximum Month Loading (MML) record of TSS in 2002 and the upper envelope is developed by factoring the 2002 AA loading of TSS by 1.38.

Figure 4.11 and 4.12a compare the upper and lower projection envelopes developed in this study based on the maximum monthly TSS parameters shown in Table 4.31a to previous TSS projection documented in the Flow and Load Projections 2001 to 2021 report (with and without TLW) (GVS&DD, 2001). The GVRD projection was developed by factoring the upper boundary of the historic AA per capita TSS load (0.124 kg/c/d).

<u>Design Case</u>

Tables 4.30b and 4.31b show the adjusted parameters for BOD (Max. Month) and TSS (Max. Month) projections under the design case based on the potential impact of the following DSM scenario: 80% of the new households will be equipped with food grinders.

The load parameters for BOD and TSS from industrial contribution are estimated to be the lower value plus 80% of the difference between the upper and lower values. Loadings are projected to grow at the same rate as population growth under the design case.

Figures 4.9b and 4.11b compare the BOD and TSS design case projections in relation to the upper and lower projection envelopes, including contribution from the Trucked Liquid Waste source.

TABLE 4.29A
AVERAGE DRY WEATHER FLOW (ADWF) PROJECTIONS AT IIWWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF ADWF PROJECTIONS UP TO YEAR 2036

	Historic data (1) **ADWF (704 LCPD)**		**Design criteria**			**Growth Rate**		
Flow Category	**Distribution (2002)**	**Flow (MLD)**	**Base Case**	**Lower & Upper Envelope**	**Remarks**	**Base Case**	**Lower & Upper Envelope**	**Remarks**
Groundwater Infiltration	32%	140	140 MLD	126 MLD	10% reduction achieved over 20 years by sewer repairs	N/a	N/a	
				147 MLD	Condition of older sewers deteriorate resulting in increased infiltration (+5%)		N/a	
Residential	38%	168	270 L/c/day (2)	214 Lcpd (2021) (3)	Reduction through implementation of water conservation - Enhanced scenario (as per memo June 18 by Clive Chapple, GRVD P&P)	In accordance with GVRD population growth (5) (720,522 by year 2021 and 755,000 by year 2036)	0.63% (700,000 by year 2021); 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (5)
				270 Lcpd	Assumes no source control		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (5)
C & I	24% (see Note: 7)	103 (see Note: 7)	166 L/c/day	153 Lcpd (2021) (3)	Reduction through implementation of water conservation - Enhanced scenario (as per memo June 18 by Clive Chapple, GRVD P&P)	In accordance with GVRD population growth (5) (720,522 by year 2021 and 755,000 by year 2036)	0.63% (700,000 by year 2021); 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (5)
				103 MLD	Assumes no source control		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (5)
Industry (6)	6%	25 (see Note: 8)	25 MLD	25 MLD	Assumes user fee policy continue in existing and new business (4)	In accordance with GVRD population growth (5) (720,522 by year 2021 and 755,000 by year 2036)	New business at 0.63% growth; Existing business at 50% of population growth (4)	New business: In accordance with population growth developed by Regional Development Division; Improvements in existing business practice
				25 MLD	Assumes a shift in industry type to biotechnology or high technology, which generate no more flow than existing industry (processing/ manufacturing) (6)		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (5) (6)
Total	100%	436						

Note:
(1) VSA Summary 1995-2002
(2) 1995 Wastewater Inventory
(3) Memo: DSM Drinking Water Conservation (dated June 18, 2003)
(4) Memo: Industry Demand Reduction Scenario - VSA 2002
(5) Current GMS 4.0 (Memo: Robert Hicks)
(6) Comments from GVRD at Workshop #1
(7) Email: Historic C&I sector flow data for Iona Island and Lions Gate WWWTP facilities planning process (original message – Clive Chapple, July 02, 2003)
(8) Revised based on (5 x 52) operating days a year

TABLE 4.29B
AVERAGE DRY WEATHER FLOW (ADWF) PROJECTIONS AT IIWWTP
DESIGN CASE SCENARIO

	Historic data ADWF (704 LCPD)		Design criteria			Growth Rate		
Flow Category	**Distribution (2002)**	**Flow (MLD)**	**Base Case**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most probable worst case scenario**
Groundwater Infiltration	32%	140	140 MLD	126 MLD	140 MLD	N/a	N/a	80% of the difference between lower and upper envelopes
				147 MLD			N/a	
Residential	38%	168	270 L/c/day	214 Lcpd (2021)	220 Lcpd (2021) ***see note 1***	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
				264 Lcpd			1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	
C & I	24%	103	166 L/c/day	153 Lcpd (2021)	166 Lcpd (2021) ***see note 1***	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
				166 Lcpd			1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	
Industry (6)	6%	25	25 MLD	25 MLD	25 MLD	720,522 by year 2021 755,000 by year 2036	New business at 0.63% growth; Existing business at 50% of population growth	0.93 % by year 2021 0.19% % by year 2036
				25 MLD			1.0% by year 2021 0.2% by year 2036	
Total	100%	436						

Note 1:Estimated per capita wastewater flow for 2021 based on existing water conservation measures

TABLE 4.30A
MAXIMUM MONTHLY (MM) BOD PROJECTIONS AT II WWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF BOD PROJECTIONS UP TO YEAR 2036

Contributor Category	Historic AA		Base Case		Design criteria		Growth Rate		
	Distribution (2001/2002)	BOD Loading (2001)	Per capita AA	MM = 1.31xAA	Lower & Upper Envelope	Remarks	Base Case	Lower & Upper Envelope	Remarks
Residential	43% to 47% (include 2.6% food waste)	32.4 tonne/day	0.053 kg/c/d (1)	0.07 kg/c/d	0.066+0.01 kg/c/d	2036: 10% of total households still have garburator (FW assumptions as per DSM memo) (6)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
					0.07+0.01 kg/c/d	Existing garubrator discharge + Increase of future garburator use (80% of new households) (6)		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
C & I	34% to 37% (include 1.4% food waste)	25.4 tonne/day (2)	0.041 kg/c/d (2)	0.05 kg/c/d	0.048 kg/c/d	FW discharge to sewer in 2036: reduced to 10% of 2002 C&I level (6)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
					0.05 kg/c/d	Food waste discharge to sewer remains same		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
Industry	20% to 26% (3)	15.3 tonne/day (AA -2001)	0.029 kg/c/d (AA – 2002)	0.04 kg/c/d	21.3 tonne/day (MM – May 2002)	Assumes user fee policy continues in existing and new business (maybe approaching point of diminishing returns) (3)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	New business at 0.63% growth; Existing business at 50% of population growth (3)	New business: In accordance with population growth developed by Regional Development Division; Improvements in existing business practice
					23.6 tonne/day	Based on 2002 AA x 1.31 (10)		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
Surface Runoff	0% to 2%	0.9 tonne/day	0.002 kg/c/d	1.8 tonne/day	1.8 tonne/day	Loading remains the same	To remain constant as 2% of AA BOD	N/a	
					1.8 tonne/day	Loading remains the same		0.2 %	Increased precipitation of 10% by 2050; 20% increase by 2080
Trucked Liquid Waste (Domestic -raw loading) (7)	0.6% (5)	0.5 tonne/day (5)	1560 mg/L (4)	0.5 tonne/day	0.35 tonne/day	30% re-distribution to AIWWTP (9)	In accordance with GVRD population growth (8) (10)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8) (10)
					0.5 tonne/day	Assumes loading remains the same		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	
WW Total	100%	74.5 t/day	0.125	0.16 kg/c/d					
TLW (Commercial -raw loading) (7)	Non-domestic: 1.4% of WW Total (5)	1.1 tonne/day (5)	41000 mg/L (4)	1.6 tonne/day	1.6 tonne/day	Average loading of 1997, 1998 and 2001	In accordance with GVRD population growth (8) (10)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8) (10)
					1.6 tonne/day	Average loading of 1997, 1998 and 2001		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	

Note:

(1) VSA Summary 1995-2002
(2) prorated from 2002 flow – C&I Summary (updated)
(3) Memo: Industry Demand Reduction Scenario - VSA 2002
(4) Trucked Liquid Waste Pricing Strategy, 1997
(5) Trucked Liquid Waste Facility Review, 2002
(6) DSM Scenarios – BOD & TSS Loadings (CP-18-04)
(7) TLW pre-treatment facility efficiency has not been assessed
(8) Current GMS 4.0 (Memo: Robert Hicks)
(9) Municipal Summary – Estimate Volumes 01/Jan/2003 to 27/Jun/2003
(10) Comments from GVRD at Workshop #1

TABLE 4.30B
MAXIMUM MONTHLY (MM) BOD PROJECTIONS AT IIWWTP
DESIGN CASE SCENARIO

Contributor Category	Historic AA		Base Case		Design criteria		Growth Rate		
	Distribution (2001/2002)	**BOD Loading (2001)**	**Per capita AA**	**Max. Month MM = 1.31xAA**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most probable worst case scenario**
Residential	43% to 47% (include 2.6% food waste)	32.4 tonne/day	0.053 kg/c/d	0.07 kg/c/d	0.066* + 0.01** kg/c/d 0.07† + 0.01†† kg/c/d	0.07† + 0.01†† kg/c/d ***see note 1***	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036) 1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
C & I	34% to 37% (include 1.4% food waste)	25.4 tonne/day	0.041 kg/c/d	0.05 kg/c/d	0.048 kg/c/d 0.05 kg/c/d	0.05 kg/c/d	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036) 1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
Industry	20% to 26%	15.3 tonne/day	0.029 kg/c/d	0.04 kg/c/d	21.3 tonne/day 23.6 tonne/day	23.1 tonne/day ***see note 2***	720,522 by year 2021 755,000 by year 2036	New business at 0.63% growth; Existing business at 50% of population growth 1.0% by year 2021 0.2% by year 2036	0.93% 0.19%
Surface Runoff	0% to 2%	0.9 tonne/day	0.002 kg/c/d	1.8 tonne/day	1.8 tonne/day 1.8 tonne/day	1.8 tonne/day	To remain constant as 2% of AA BOD	N/a 0.2 %	0.2%
Trucked Liquid Waste (Domestic –raw loading)	0.6% (5)	0.5 tonne/day	1560 mg/L	0.5 tonne/day	0.35 tonne/day 0.5 tonne/day	0.5 tonne/day ***see note 3***	720,522 by year 2021 755,000 by year 2036	0.63% by year 2021 0.09% by year 2036 1.0% by year 2021 0.2% by year 2036	0.93% 0.19%
WW Total	100%	74.5 t/day	0.125	0.16 kg/c/d					
TLW (Commercial – raw loading)	1.4% of WW Total	1.1 tonne/day	41,000 mg/L	1.6 tonne/day	1.6 tonne/day 1.6 tonne/day	1.6 tonne/day	720,522 by year 2021 755,000 by year 2036	0.63% by year 2021 0.09% by year 2036 1.0% by year 2021 0.2% by year 2036	0.93% 0.19%

*BOD contribution from Residential sources with complete elimination of food waste discharges to sewer
**10% of total households contribute 0.01 kg/c/d BOD loads by food waste discharges to sewer
† BOD contribution from Residential sources (including food waste discharges from existing households)
†† 80% of new households contribute 0.01 kg/c/d BOD loads by food waste discharges to sewer

Note 1: 80% of new households assumed to be equipped with food grinders
Note 2: 80% of the difference between lower and upper load parameters
Note 3: No re-distribution to AIWWTP

TABLE 4.31A
MAXIMUM MONTHLY (MM) TSS PROJECTIONS AT II WWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF TSS PROJECTIONS UP TO YEAR 2036

	Average Annual (AA)		MML =1.38xAA		Design criteria		Growth Rate		
Contributor Category	**Distribution (2002)**	**BOD Loading (2002)**	**Per capita AA**	**Base Case (Max. Month)**	**Lower & Upper Envelope**	**Remarks**	**Base Case**	**Lower & Upper Envelope**	**Remarks**
Residential	56% (include 3.9% food waste)	37.7 tonne/day	0.061 kg/c/d (1)	0.08 kg/c/d	0.078 + 0.02 kg/c/d	2036: 10% of total households still have garburator (FW assumptions as per DSM memo) (6)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
					0.07 + 0.02 kg/c/d	Existing garubrator discharge + Increase of future garburator use (80% of new households) (6)		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
C & I	18% (include 2.0% food waste)	13.1 tonne/day	0.021 kg/c/d (2)	0.03 kg/c/d	0.027 kg/c/d	FW discharge to sewer in 2036: reduced to 10% of 2002 C&I level (6)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
					0.03 kg/c/d	Food waste discharge to sewer remains same		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
Industry	7% (3)	4.9 tonne/day (AA – 2002)	0.008 kg/c/d	0.01 kg/c/d	6.8 tonne/day (2002 AA x 1.38 PF)	Assumes user fee policy continues in existing and new business (maybe approaching point of diminishing returns) (3)	In accordance with GVRD population growth (720,522 by year 2021 and 755,000 by year 2036) (8)	New business at 0.63% growth; Existing business at 50% of population growth (3)	New business: In accordance with population growth developed by Regional Development Division; Improvements in existing business practice
					7.1 tonne/day	Based on 2002 Max. Month record (May) (10)		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)
Surface Runoff	17%	12.5 tonne/day	0.018 kg/c/d	15 tonne/day	15 tonne/day	Loading remains the same	To remain constant as 17% of AA TSS	N/a	
					15 tonne/day	Loading remains the same		0.2 %	Increased precipitation of 10% by 2050; 20% increase by 2080
Trucked Liquid Waste (Domestic – raw loading) (7)	2.6% (5)	1.9 tonne/day (5)	5060 mg/L (4)	1.9 tonne/day	1.3 tonne/day	30% re-distribution to AIWWTP (9)	In accordance with GVRD population growth (8)(10)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)(10)
					1.9 tonne/day	Assumes loading remains the same		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	
WW Total	100%	70 t/day	0.108	0.15 kg/c/d					
Trucked Liquid Waste (Raw loading) (7)	Non-domestic: 5.2% of WW Total (5)	3.7 tonne/day (5)	101000 mg/L (4)	4.0 tonne/day	4.0 tonne/day	Average loading of 1997, 2000 and 2001	In accordance with GVRD population growth (8)(10)	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036)	In accordance with population ranges developed by Regional Development Division (8)(10)
					4.0 tonne/day	Average loading of 1997, 2000 and 2001		1.0% (750,000 by year 2021); 0.2% (775,000 by year 2036)	

Note:
(1) VSA Summary 1995-2002
(2) prorated from 2002 flow – C&I Summary (updated)
(3) Memo: Industry Demand Reduction Scenario - VSA 2002
(4) Trucked Liquid Waste Pricing Strategy, 1997
(5) Trucked Liquid Waste Facility Review, 2002
(6) DSM Scenarios – BOD & TSS Loadings (CP-18-04)
(7) TLW pre-treatment facility efficiency has not been assessed
(8) Current GMS 4.0 (Memo: Robert Hicks)
(9) Municipal Summary – Estimate Volumes 01/Jan/2003 to 27/Jun/2003
(10) Comments from GVRD at Workshop #1

TABLE 4.31B
MAXIMUM MONTHLY (MM) TSS PROJECTIONS AT IIWWTP
DESIGN CASE SCENARIO

	Average Annual (AA)		**Base Case**		**Design criteria**		**Growth Rate**		
Contributor Category	**Distribution (2002)**	**BOD Loading (2002)**	**Per capita AA**	**Max. Month MM=1.38xAA**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most probable worst case scenario**
Residential	56% (include 3.9% food waste)	37.7 tonne/day	0.061 kg/c/d	0.08 kg/c/d	0.078* + 0.02** kg/c/d 0.08† + 0.02†† kg/c/d	0.08† + 0.02†† kg/c/d ***see note 1***	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036) 1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
C & I	18% (include 2.0% food waste)	13.1 tonne/day	0.021 kg/c/d	0.03 kg/c/d	0.027 kg/c/d 0.03 kg/c/d	0.03 kg/c/d	720,522 by year 2021 755,000 by year 2036	0.63% (700,000 by year 2021) 0.09% (710,000 by year 2036) 1.0% (750,000 by year 2021) 0.2% (775,000 by year 2036)	0.93% (740,000 by year 2021) 0.19% (760,000 by year 2036)
Industry	7%	4.9 tonne/day	0.008 kg/c/d	0.01 kg/c/d	6.8 tonne/day 7.1 tonne/day	7.0 tonne/day ***see note 2***	720,522 by year 2021 755,000 by year 2036	New business at 0.63% growth; Existing business at 50% of population growth 1.0% by year 2021 0.2% by year 2036	0.93% by year 2021 0.19% by year 2036
Surface Runoff	17%	12.5 tonne/day	0.018 kg/c/d	15 tonne/day	15 tonne/day 15 tonne/day	15 tonne/day	To remain constant as 17% of AA TSS	N/a 0.2 %	0.2%
Trucked Liquid Waste (Domestic – raw loading)	2.6%	1.9 tonne/day	5060 mg/L	1.9 tonne/day	1.3 tonne/day 1.9 tonne/day	1.9 tonne/day ***see note 3***	720,522 by year 2021 755,000 by year 2036	0.63% by year 2021 0.09% by year 2036 1.0% by year 2021 0.2% by year 2036	0.93% by year 2021 0.19% by year 2036
WW Total	100%	70 t/day	0.108	0.15 kg/c/d					
Truck Liquid Waste (Commercial – raw loading)	5.2% of WW Total	3.7 tonne/day	101,000 mg/L	4.0 tonne/day	4.0 tonne/day 4.0 tonne/day	4.0 tonne/day	720,522 by year 2021 755,000 by year 2036	0.63% by year 2021 0.09% by year 2036 1.0% by year 2021 0.2% by year 2036	0.93% by year 2021 0.19% by year 2036

*TSS contribution from Residential sources with complete elimination of food waste discharges to sewer
**10% of total households contribute 0.02 kg/c/d TSS loads by food waste discharges to sewer
† TSS contribution from Residential sources (including food waste discharges from existing households)
†† 80% of new households contribute 0.02 kg/c/d TSS loads by food waste discharges to sewer

Note 1: 80% of the new households assumed to be equipped with food grinders
Note 2: 80% of the difference between lower and upper load parameters
Note 3: No re-distribution to AIWWTP

FIGURE 4.5
ADWF ENVELOPE BY SECTOR – IIWWTP
(RESIDENTIAL, C&I, INDUSTRY, INFILTRATION)

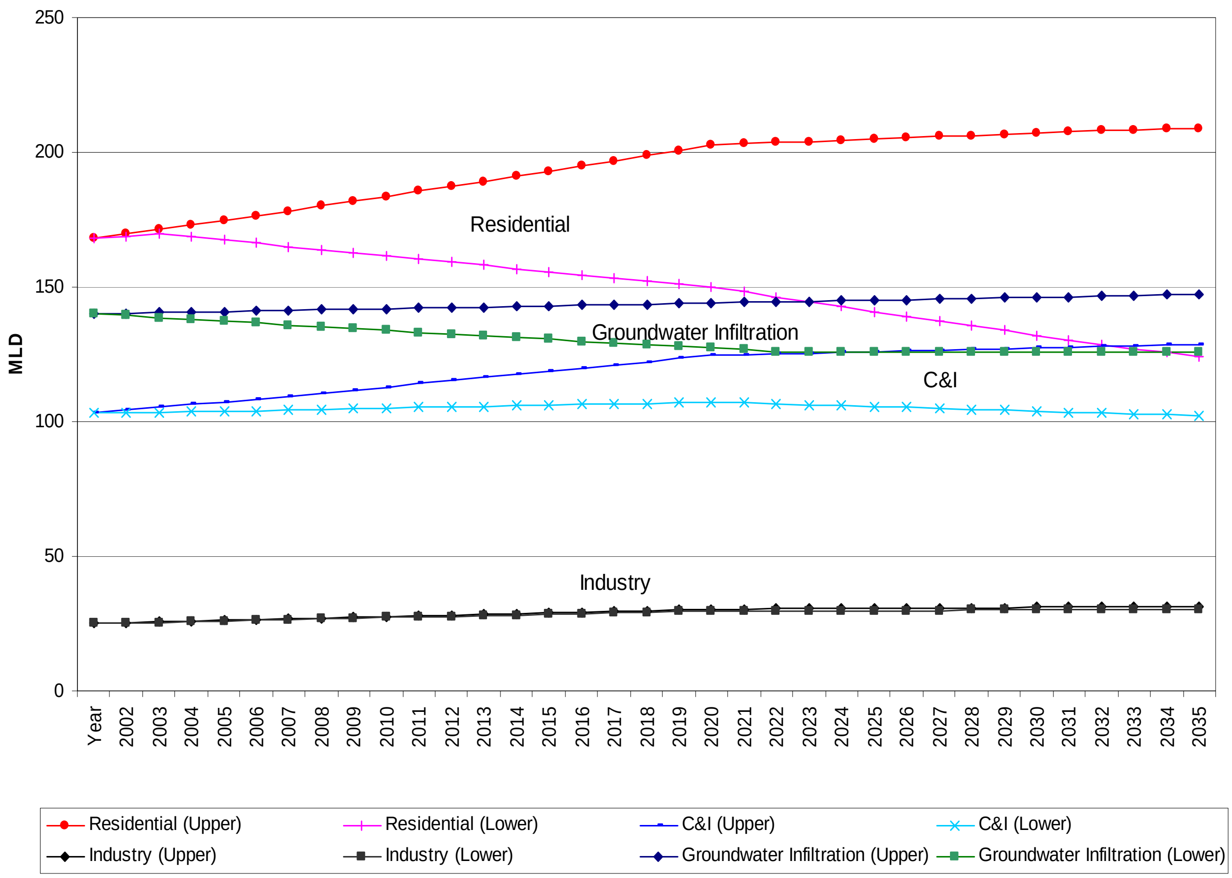


FIGURE 4.6a
IIWWTP UPPER AND LOWER PROJECTION ENVELOPE FOR ADWF

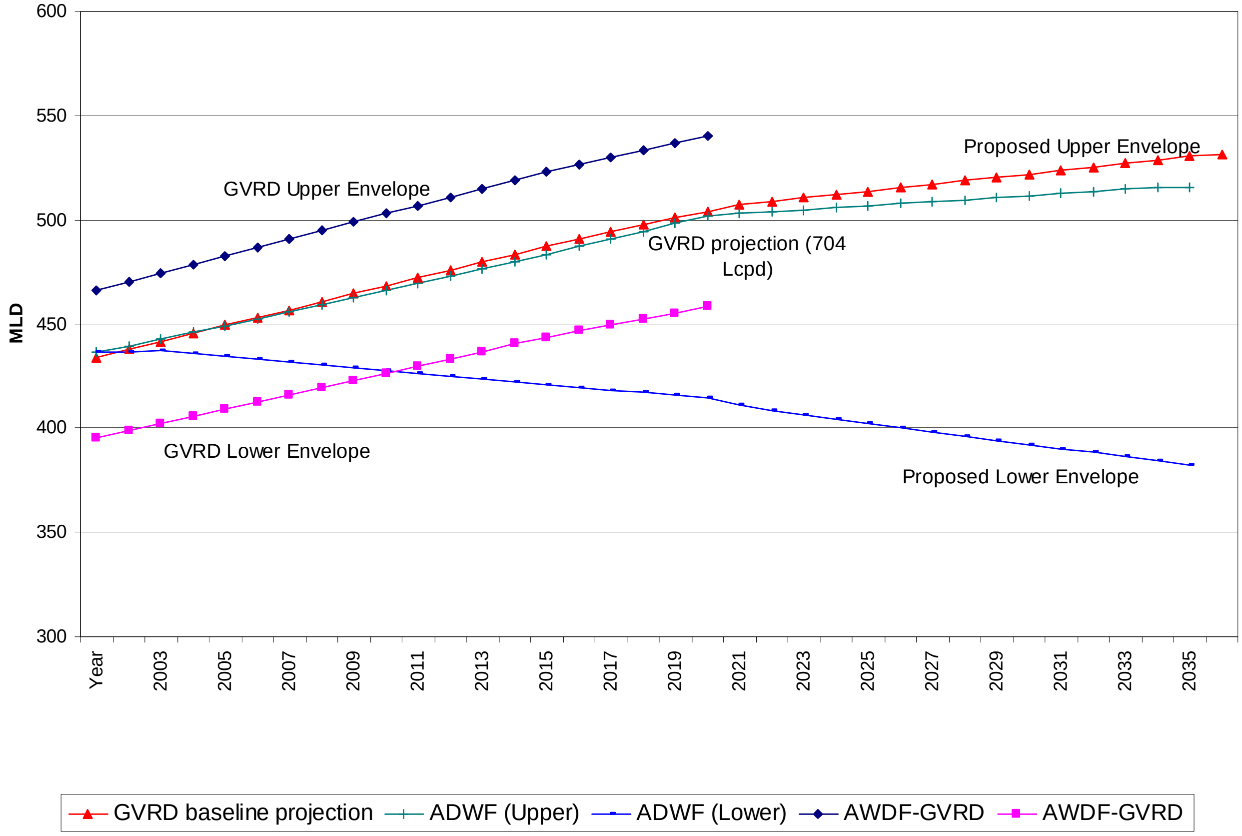


FIGURE 4.6b
IIWWTP DESIGN CASE SCENARIO FOR ADWF

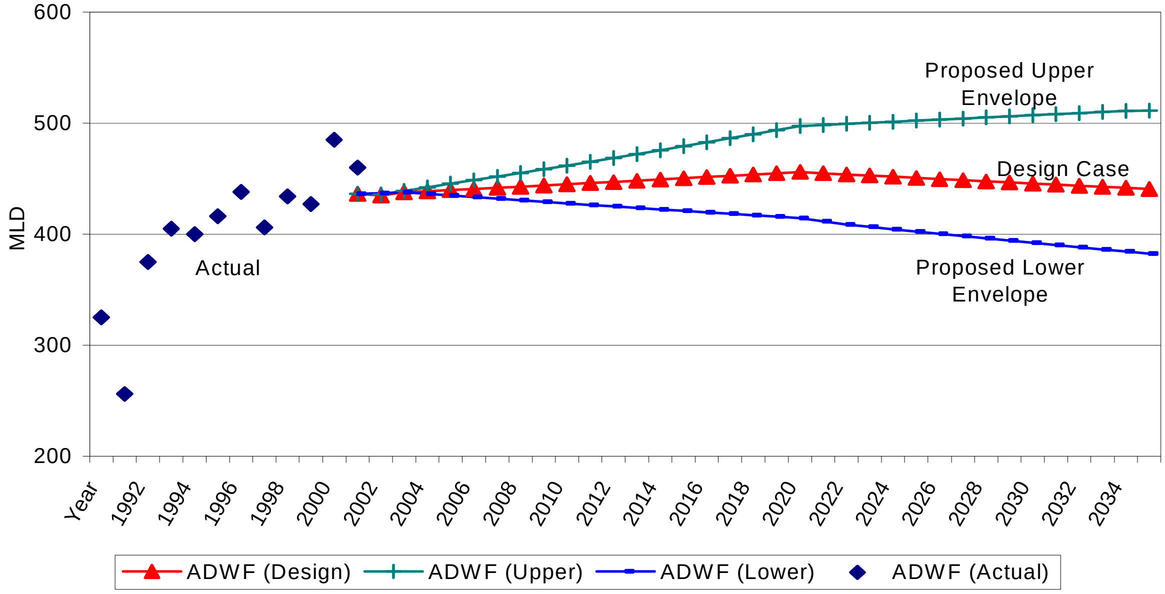


FIGURE 4.7
BOD MAX. MONTH (MM) ENVELOPE PROJECTION BY SECTOR – IIWWTP
(RESIDENTIAL C&I, INDUSTRY, TLW)

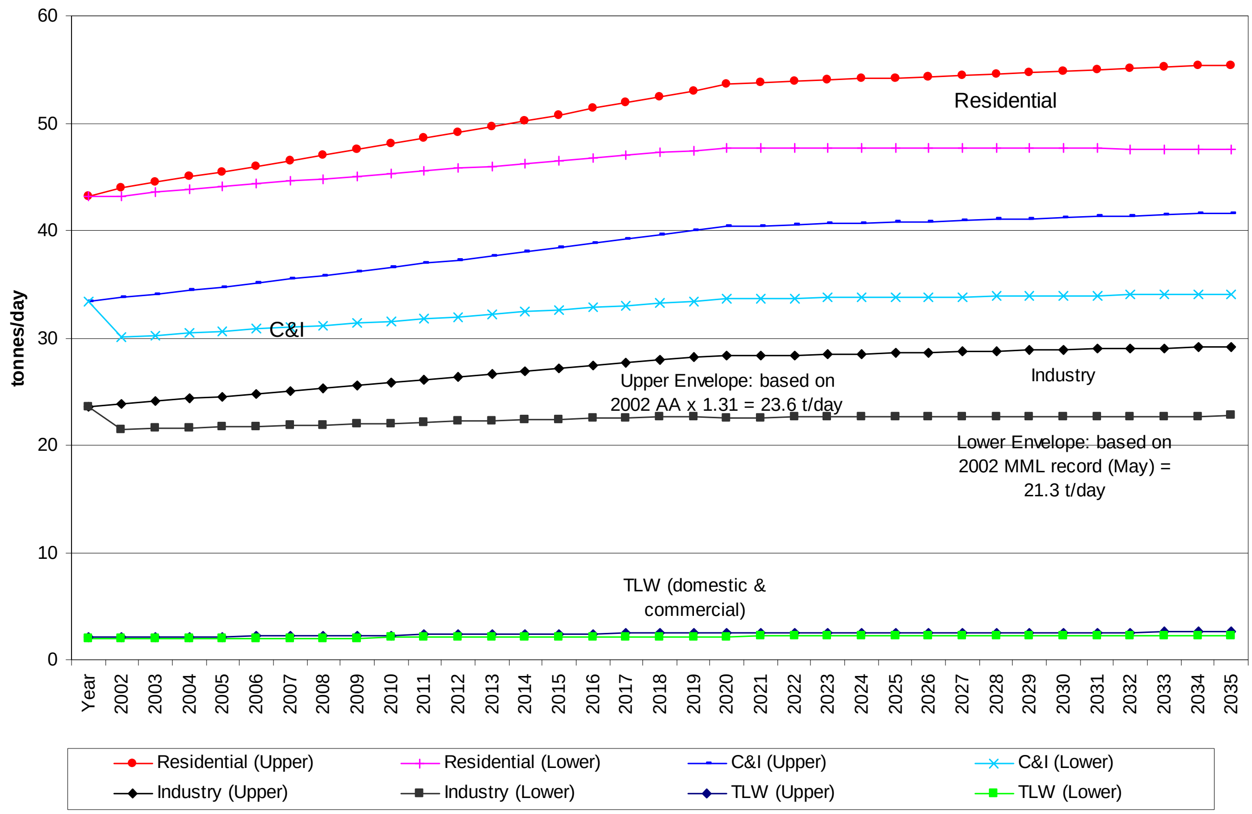


FIGURE 4.8
MAX. MONTH (MM) BOD UPPER & LOWER PROJECTION ENVELOPE – IIWWTP
(TRUCKED LIQUID WASTE NOT INCLUDED)

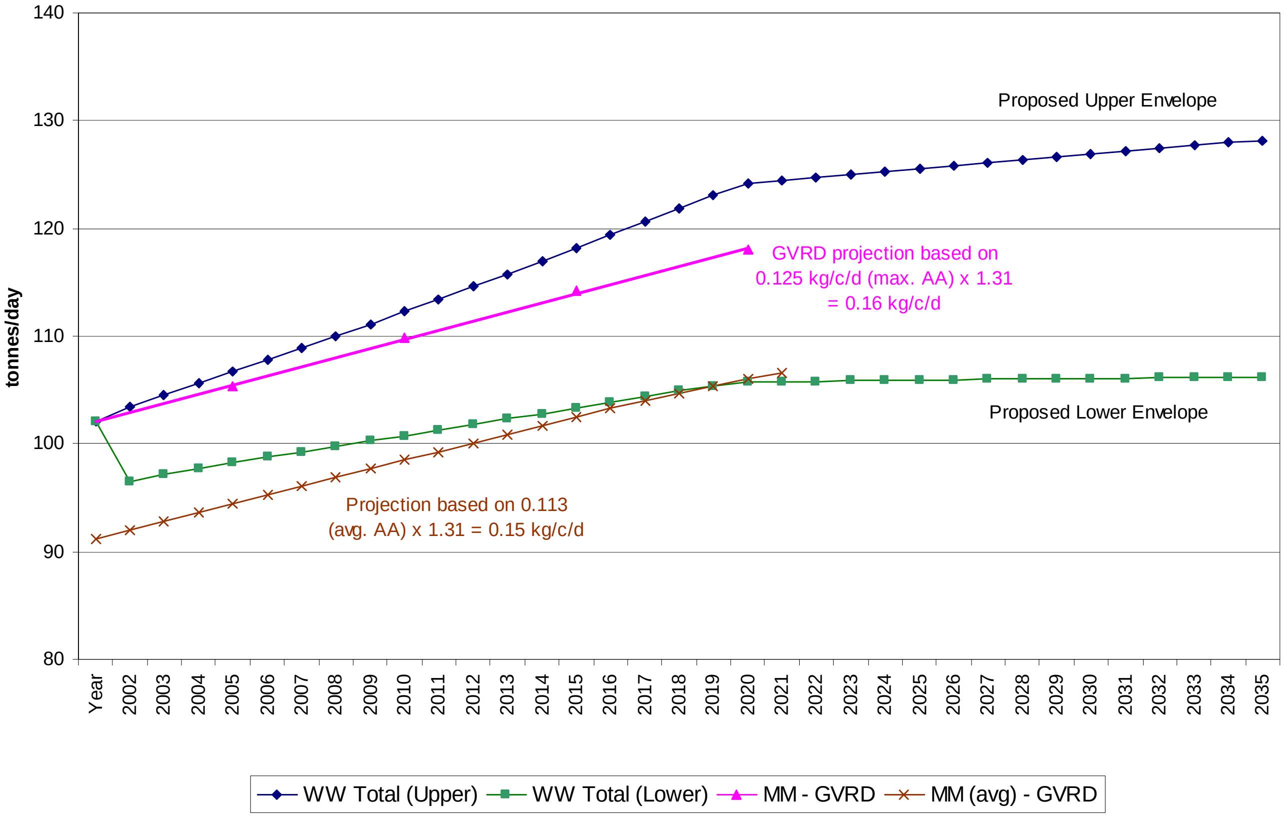


FIGURE 4.9a
MAX. MONTH (MM) BOD UPPER & LOWER PROJECTION ENVELOPE – IIWWTP
(TRUCKED LIQUID WASTE INCLUDED)

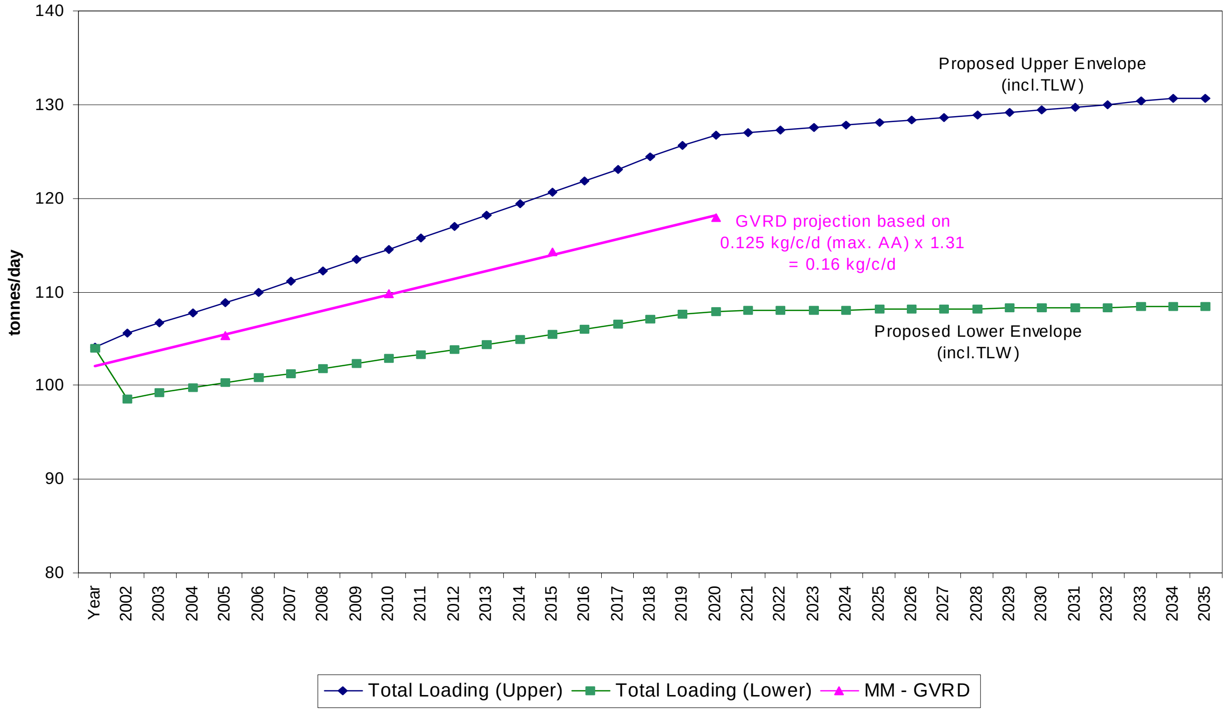


FIGURE 4.9b
MAX. MONTH (MM) BOD UPPER & LOWER PROJECTION ENVELOPE – IIWWTP (TRUCKED LIQUID WASTE INCLUDED)

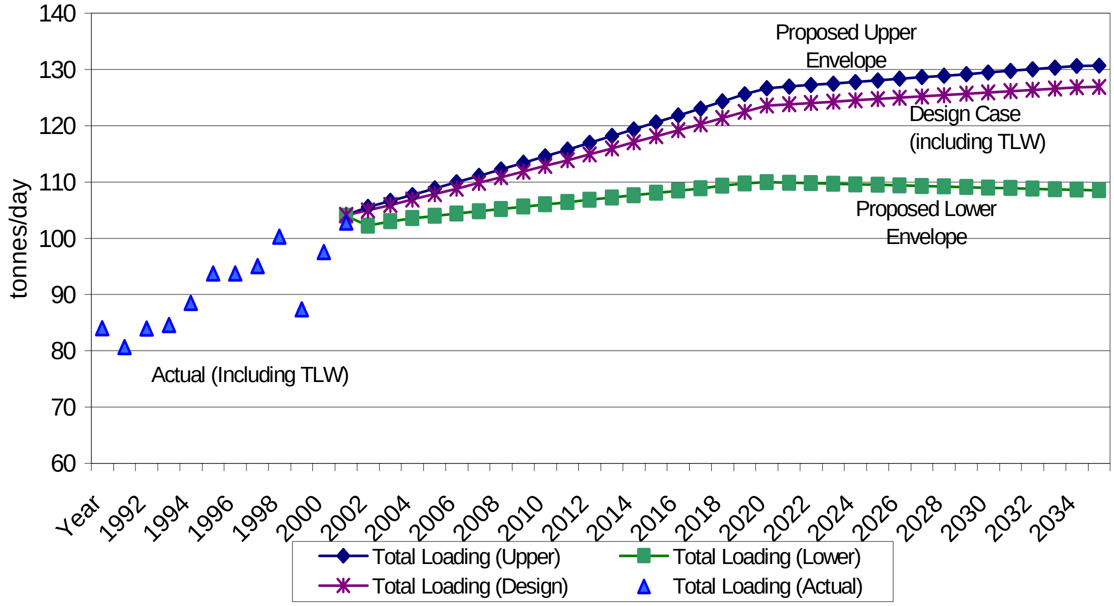


FIGURE 4.10
TSS MAX. MONTH (MM) ENVELOPE PROJECTION BY SECTOR – IIWWTP
(RESIDENTIAL C&I, INDUSTRY, TLW)

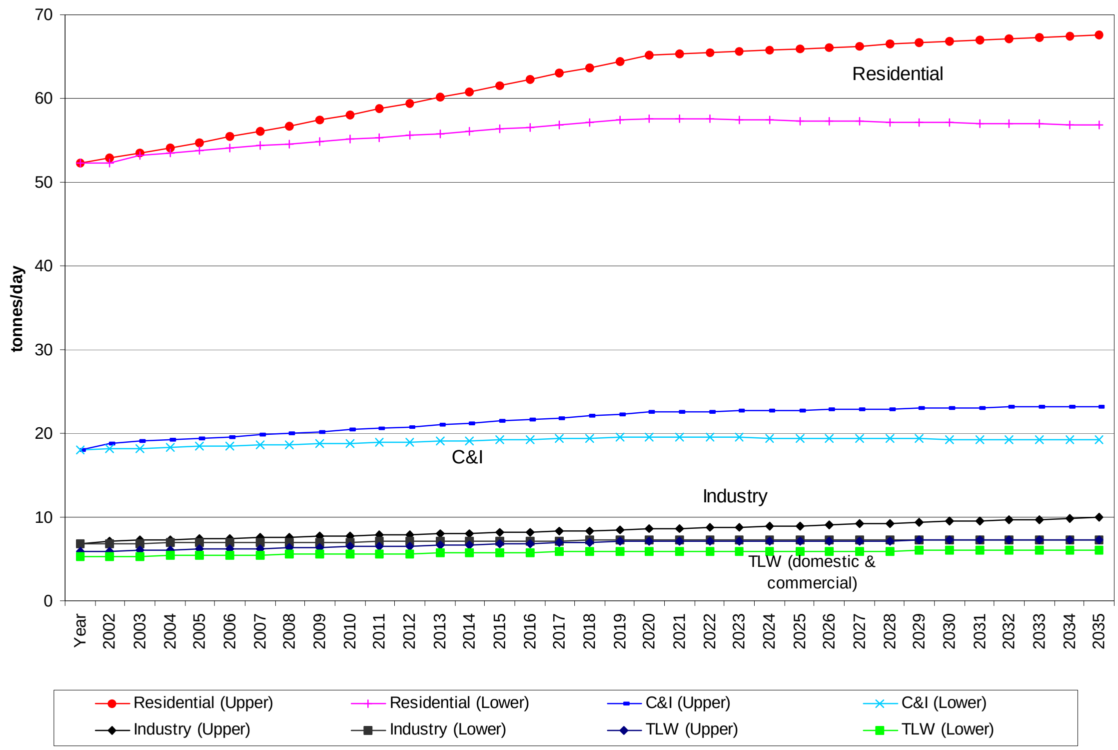


FIGURE 4.11
MAX. MONTH (MM) TSS UPPER & LOWER PROJECTION ENVELOPE – IIWWTP (TRUCKED LIQUID WASTE NOT INCLUDED)

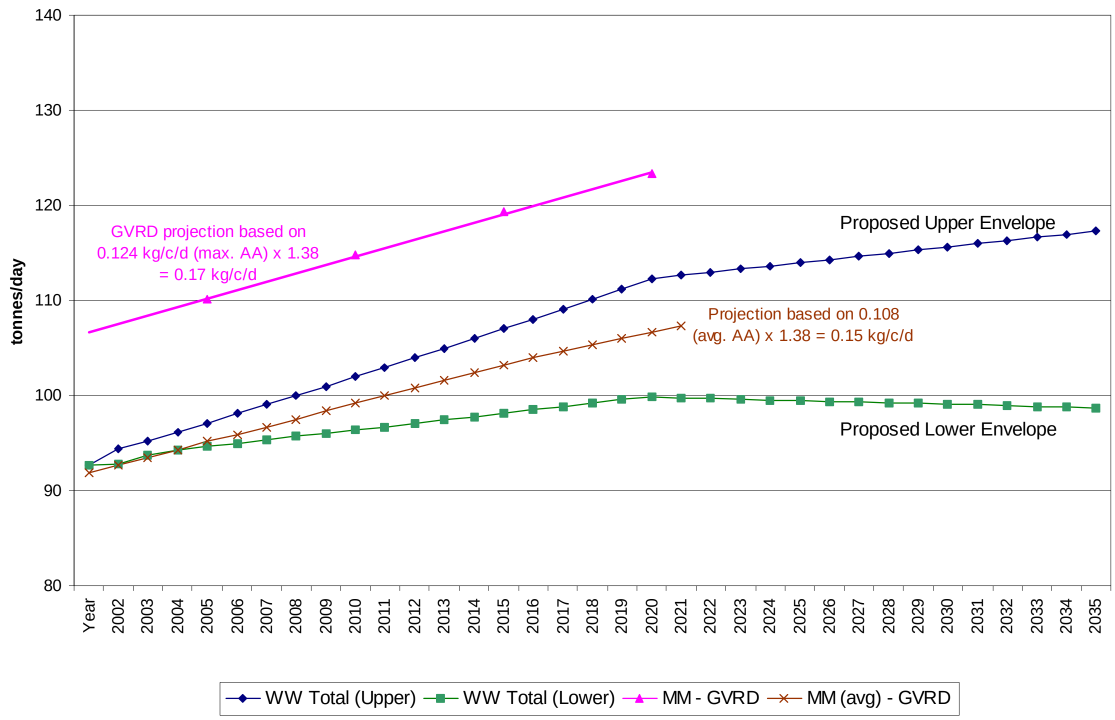


FIGURE 4.12a
MAX. MONTH (MM) TSS UPPER & LOWER PROJECTION ENVELOPE – IIWWTP (TRUCKED LIQUID WASTE INCLUDED)

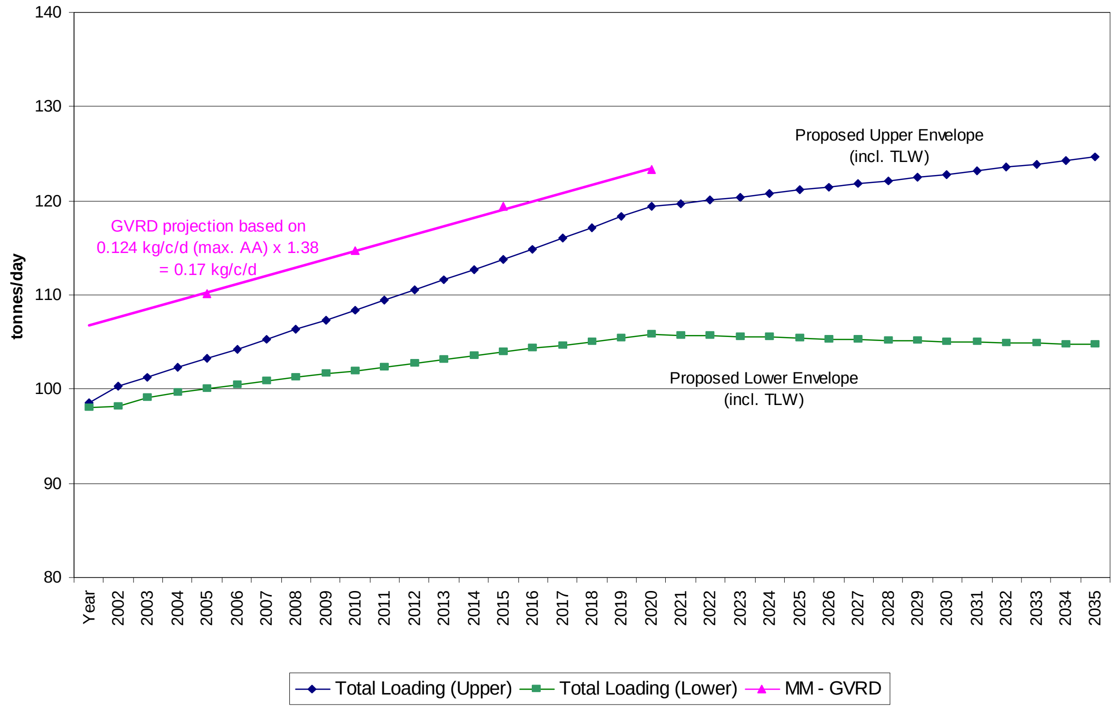
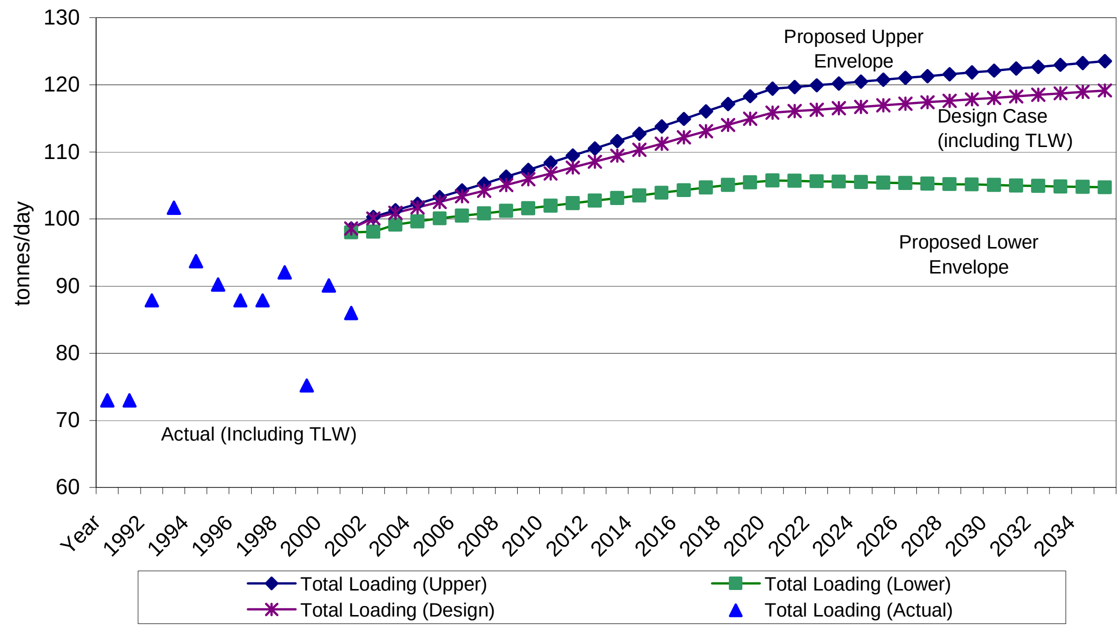


FIGURE 4.12b
MAX. MONTH (MM) TSS UPPER & LOWER PROJECTION ENVELOPE – IIWWTP



4.8.2 Lions Gate WWTP

A spreadsheet system has been developed for LGWWTP to project the population in the NSSA to year 2021 (design year for interim upgrade), 2046 (design year for build-out to secondary) and 2081 (design year for sewerage system). The flow and load by sector discharging to the LGWWTP are projected to the design years based on historical data as well as comments and memos given by the GVRD. Graphical representations of upper envelope and lower envelope are generated based on the spreadsheet projection. They will be presented in the following sections. Summarized tables presenting historical data of the base year 2002 and design criteria for upper and lower envelope for flow and load in the NSSA are also included in the following sections. Table 4.32a, 4.33a and 4.34a summarize the design criteria for ADWF, BOD and TSS respectively in the base case and upper and lower projection envelopes. Table 4.32b, 4.33b and 4.34b illustrate the counterparts for design case.

4.8.2.1 Flow - ADWF

Upper and Lower Envelope

Table 4.32a summarizes the factors affecting upper and lower envelope of Average Dry Weather Flow (ADWF) projections up to year 2046 for LGWWTP. Demand Side Management (DSM) initiatives as well as source control on flow discussed in Section

4.5 are incorporated into the spreadsheet as parameters for developing upper and lower envelope.

In general, flow projection by sector is calculated based on historical data in 2002 (base year) multiplied by projected population. Impacts of flow control are taken into consideration as percentage reduction of flow in projecting the upper and lower envelopes. Flow achieved by reduction initiatives is shown as 'Design Criteria' under the 'Lower & Upper Envelope' column in Table 4.32. On the other hand, population is projected by calculated growth rate based on population given in the GVRD Flow and Load Projections Report 2001 to 2021 (GVRD, 2001b) and Growth Management Scenario 4.0 in 5-year, 20-year and 50-year intervals (Section 4.2).

The ADWF projected envelopes by Sector for LGWWTP from 2002 to 2046 are presented in Figure 4.13. The upper and lower envelopes of flows contributed from Residential, Commercial & Institutional (C&I), Industrial and Groundwater Infiltration are shown in the figure. Figure 4.14a shows the combined upper and lower envelopes. The projections are based on per capita ADWF values given by GVRD from 2002 to 2046. The upper envelope is projected with the use of 566 L/c/d, which is the upper boundary of the ADWF envelope based on the average unit flows for 1991 to 1999 inclusive. Similarly, the lower envelope is projected with the use of 464 L/c/d, which is the lower boundary of the ADWF envelope based on the average unit flows for 1991 to 1999 inclusive. The GVRD projection is based on the use of per capita ADWF of 518 L/c/d.

Proposed upper envelope and lower envelope of total flow from this study are also shown in Figure 4.14. It is worth noting that the proposed upper envelope developed in this study based on source contribution is within the GVRD projection envelopes.

Design case

Table 4.32b summarizes the adjusted key parameters for ADWF under the design case. Figure 4.14b compares the design case with the upper and lower projection envelope. The ADWF in the design case approaches 104 ML/d in year 2031.

4.8.2.2 Load

Upper and Lower Projection Envelopes

Table 4.33a and 4.34a summarize the factors affecting the upper and lower envelope of BOD and TSS projections up to year 2046 for LGWWTP, respectively. Similar to the flow projection, Demand Side Management (DSM) initiatives as well as source control on loading discussed in Section 4.5 are incorporated into the spreadsheet as parameters for developing the upper and lower envelopes. They are summarized as "remarks" under the column 'Design Criteria' in the table.

Generally, load projection by sector is calculated based on historical data in 2002 (base year) multiplied by projected population. Impacts of load control are taken into consideration as percentage reduction of load by sector in projecting the upper and lower envelopes. BOD and TSS achieved by reduction initiatives are shown as

'Design Criteria' under the 'Lower & Upper Envelope' column in Table 4.33a and 4.34a, respectively. Maximum month (MM) loading is used instead of Average Annual (AA) loading. MM loading is calculated by factoring the AA loading by 1.34 for BOD and 1.43 for TSS.

In Table 4.33a and 4.34a, it is noted that design criteria for lower and upper envelopes of the residential sector are represented by a combination of two numbers. In the lower envelope, the first number (0.067 kg/c/d for BOD and 0.081 kg/c/d for TSS) represents per capita loading without the use of garburators, while the second number (0.001 kg/c/d for BOD and 0.002 kg/c/d for TSS) represents the per capita loading contributed only by food waste from the use of garburators. The second number only is based on a reduction by 2046 to 10% of the current use of garburators, which is assumed to be 33% of the total households (Compass, 1998). While in the upper envelope, the first number (0.071 kg/c/d for BOD and 0.09 kg/c/d for TSS) represents loading with existing garburator discharge, the second number (0.001 kg/c/d for BOD and 0.002 kg/c/d for TSS) represents the per capita loading contributed only by food waste from the use of garburators. The increase of BOD and TSS due to increase in garburator use in the future, which is 80% of the new households, will be calculated based on the second number.

Population is projected by calculated growth rate based on population given in the GVRD Flow and Load Projections Report 2001 to 2021 and Growth Management Scenario 4.0 in 5-year, 20-year and 50-year intervals (Section 4.2).

Figure 4.15 and Figure 4.17 show the loading envelope projection for BOD (MM) and TSS (MM), respectively. Loadings contributed from Residential, Commercial & Institutional (C&I), Industrial sectors are shown in these figures. These are graphical representations of the projection data calculated from the spreadsheet system.

It should be noted that the upper envelope is developed based on Maximum Month Loading (MML) record of BOD in 2002 whereas the lower envelope is developed by factoring the 2002 AA loading of BOD by 1.34. The lesser value between the actual BOD maximum monthly record and the baseline maximum monthly BOD derived from historic factor is used for lower envelope projection (Section 4.5.4.5).

Figure 4.16a and Figure 4.18a show the upper and lower envelopes for BOD and TSS based on per capita daily loading given by the GVRD from 2002 to 2046. In Figure 4.16a, the upper envelope for BOD is projected with the use of 0.115 kg/c/d, which is calculated by factoring the upper boundary of the average annual per capita BOD contribution based on the average unit loads for 1991 to 1999 inclusive (0.086 kg/c/d) by 1.34. On the other hand, the lower envelope is projected with the use of 0.103 kg/c/d, which is calculated by factoring the average value of the average unit loads for 1993 to 1999 inclusive (0.77 kg/c/d) by 1.34. Proposed upper envelope and lower envelope of total BOD from this study is also shown in Figure 4.16a. The upper envelope developed from this study stays within the GVRD envelope until year 2016.

In Figure 4.18a, the upper envelope for TSS is projected with the use of 0.150 kg/c/d, which is calculated by factoring the upper boundary of the average annual per capita

TSS contribution based on the average unit loads for 1991 to 1999 inclusive (0.104 kg/c/d) by 1.43. Similarly, the lower envelope is projected with the use of 0.130 kg/c/d, which is calculated by factoring the average value of the average unit loads for 1993 to 1999 inclusive (0.088 kg/c/d) by 1.43. Proposed upper envelope and lower envelope of total BOD from this study is also shown in Figure 4.18. The upper envelope developed from this study stays within the GVRD envelope until year 2030.

<u>Design Case Scenario</u>

Tables 4.33b and 4.34b show the adjusted parameters for BOD (Max. Month) and TSS (Max. Month) projections under the design case based on the potential impacts of the following assumptions:

- The proportion of households equipped with food grinders will be increased from 33% (2002) to 38.2% (2021) and further to 46.7% (2046) for NSSA; this will affect residential and C&I loadings.
- The load parameters for BOD and TSS from industrial contribution are estimated to be lower envelope plus 80% of the difference between the upper and lower values. Loadings are projected to grow at the same rate as population growth under the design case.

Figures 4.16b and 4.18b compare the BOD and TSS projection under the design case with the upper and lower envelope.

TABLE 4-32A
AVERAGE DRY WEATHER FLOW (ADWF) PROJECTIONS AT LGWWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF ADWF PROJECTIONS UP TO YEAR 2046

<table>
<tr><th rowspan="2"></th><th colspan="2">Historic data (1)</th><th colspan="3" rowspan="2">Design criteria</th><th colspan="3" rowspan="2">Growth Rate</th></tr>
<tr><th colspan="2">ADWF (518 LCPD)</th></tr>
<tr><th>Flow Category</th><th>Distribution (2002)</th><th>Flow (MLD)</th><th>Base Case</th><th>Lower & Upper Envelope</th><th>Remarks</th><th>Base Case</th><th>Lower & Upper Envelope</th><th>Remarks</th></tr>
<tr><td rowspan="2">Groundwater Infiltration</td><td rowspan="2">31%</td><td rowspan="2">28</td><td rowspan="2">28 MLD</td><td>25 MLD</td><td>10% reduction achieved over 20 years by sewer repairs</td><td rowspan="2">N/a</td><td>N/a</td><td></td></tr>
<tr><td>29 MLD</td><td>Condition of older sewers deteriorate resulting in increased infiltration (+5%)</td><td>N/a</td><td></td></tr>
<tr><td rowspan="2">Residential</td><td rowspan="2">52%</td><td rowspan="2">47</td><td rowspan="2">270 Lpcd (2)</td><td>232 Lcpd (2021) (3)</td><td>Reduction through implementation of water conservation - Enhanced scenario (as per memo June 18 by Clive Chapple, GVRD P&P)</td><td rowspan="2">In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2046)</td><td>0.71% (200,000 by year 2021);
0.75% (241,200 by year 2046)</td><td>In accordance with population ranges developed by Regional Development Division (5)</td></tr>
<tr><td>270 Lcpd</td><td>Assumes no source control</td><td>1.19% (220,000 by year 2021);
1.04% (283,600 by year 2046)</td><td>In accordance with population ranges developed by Regional Development Division (5)</td></tr>
<tr><td rowspan="2">C & I</td><td rowspan="2">11%
(see Note: 7)</td><td rowspan="2">9.7
(see Note: 7)</td><td rowspan="2">55 Lpcd</td><td>51 Lcpd (2021) (3)</td><td>Reduction through implementation of water conservation - Enhanced scenario (as per memo June 18 by Clive Chapple, GVRD P&P)</td><td rowspan="2">In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2046)</td><td>0.71% (200,000 by year 2021);
0.75% (241,200 by year 2046)</td><td>In accordance with population ranges developed by Regional Development Division (5)</td></tr>
<tr><td>9.7 MLD</td><td>Assumes no source control</td><td>1.19% (220,000 by year 2021);
1.04% (283,600 by year 2046)</td><td>In accordance with population ranges developed by Regional Development Division (5)</td></tr>
<tr><td rowspan="2">Industry
(6)</td><td rowspan="2">7%</td><td rowspan="2">6</td><td rowspan="2">6 MLD</td><td>6 MLD</td><td>Assumes user fee policy continue in existing and new business
(4)</td><td rowspan="2">In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2046)</td><td>New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; (5)
Existing business at 50% of population growth
(4)</td><td>New business: In accordance with population growth developed by Regional Development Division;
Improvements in existing business practice</td></tr>
<tr><td>6 MLD</td><td>Assumes a shift in industry type to biotechnology or high technology, which generate no more flow than existing industry (processing/ manufacturing) (6)</td><td>1.19% (220,000 by year 2021);
1.04% (283,600 by year 2046)</td><td>In accordance with population ranges developed by Regional Development Division
(5) (6)</td></tr>
<tr><td>Total</td><td>100%</td><td>90</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

Note:
(1) NSSA Summary 1995-2002 (2) 1995 Wastewater Inventory (3) Memo: DSM Drinking Water Conservation (dated June 18, 2003)
(4) Memo: Industry Demand Reduction Scenario - NSSA 2002 (5) Current GMS 4.0 (Memo: Robert Hicks) (6) Comments from GVRD at Workshop #1
(7) Email: Historic C&I sector flow data for Iona Island and Lions Gate WWWTP facilities planning process (original message – Clive Chapple, July 02, 2003)

TABLE 4-32B
AVERAGE DRY WEATHER FLOW (ADWF) PROJECTIONS AT LGWWTP
DESIGN CASE SCENARIO

	Historic data ADWF (518 LCPD)		Design criteria			Growth Rate		
Flow Category	**Distribution (2002)**	**Flow (MLD)**	**Base Case**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most probable worst case scenario**
Groundwater Infiltration	31%	28	28 MLD	25 MLD 29 MLD	28 MLD	N/a	N/a N/a	80% of the difference between lower and upper envelope
Residential	52%	47	270 Lpcd (2)	232 Lcpd (2021) (3) 270 Lcpd	243 Lpcd (1)	196,765 by year 2021 248,000 by year 2046	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046) 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
C & I	11% (see Note: 7)	9.7 (see Note: 7)	55 Lpcd	51 Lcpd (2021) (3) 9.7 MLD	56 Lpcd (1)	196,765 by year 2021 248,000 by year 2046	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046) 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
Industry (6)	7%	6	6 MLD	6 MLD 6 MLD	6 MLD	196,765 by year 2021 248,000 by year 2046	New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; Existing business at 50% of population growth 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
Total	100%	90						

Note:

(1) Estimated per capita wastewater flow for 2021 based on existing water conservation measures

TABLE 4.33A
MAXIMUM MONTHLY (MM) BOD PROJECTIONS AT LGWWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF BOD PROJECTIONS UP TO YEAR 2046

Contributor Category	Historic AA		Base Case		Design criteria		Growth Rate		
	Distribution (2001/2002)	**BOD Loading (2001)**	**Per capita AA**	**MM = 1.34xAA**	**Lower & Upper Envelope**	**Remarks**	**Base Case**	**Lower & Upper Envelope**	**Remarks**
Residential	77 to 69% (include 4.2% food waste)	9.2 tonne/day	0.053 kg/c/d (1)	0.071 kg/c/d	0.067+0.01 kg/c/d	2046: 10% of total households still have garburator (Food Waste assumptions as per DSM memo) (4)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
					0.071+0.01 kg/c/d	Existing garburator discharge + increase of future garburator use (80% of new households) (4)		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
C & I	21 to 18% (include 1.5% food waste)	2.44 tonne/day	0.014 kg/c/d (2)	0.02 kg/c/d	0.019 kg/c/d	Food Waste discharge to sewer in 2046: reduced to 10% of 2002 C&I level (4)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
					0.02 kg/c/d	Food waste discharge to sewer remains the same		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
Industry	3% unchanged (3)	0.3 tonne/day (AA-2001)	0.002 kg/c/d (AA-2002)	0.003 kg/c/d	0.54 tonne/day (2002 AA x 1.34 PF)	Assumes user fee policy continues in existing and new business (maybe approaching point of diminishing returns) (3)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; Existing business at 50% of population growth (3)	New business: In accordance with population growth developed by Regional Development Division; Improvements in existing business practice
					1.11 tonne/day	Based on 2002 Max. Month record (August) (6)		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
Surface Runoff	0 to 10%	0 tonne/day	0.008 kg/c/d	1.8 tonne/day	1.9 t/day	Loading remains the same	To remain constant as 10% AA BOD	N/A	
					1.9 t/day	Loading remains the same		0.2% by year 2050, 0.3% from 2051 to 2080	Increased precipitation of 10% by 2050; 20% increase by 2080
WW Total	100%	11.9 t/day	0.077	0.1 kg/c/d					

Note:
(1) NSSA Summary 1995-2002
(2) prorated from 2002 flow – C&I Summary (updated)
(3) Memo: Industry Demand Reduction Scenario - NSSA 2002
(4) DSM Scenarios – BOD & TSS Loadings (CP-18-04)
(5) Current GMS 4.0 (Memo: Robert Hicks)
(6) Comments from GVRD at Workshop #1

TABLE 4.33B
MAXIMUM MONTHLY (MM) BOD PROJECTIONS AT LGWWTP
DESIGN CASE SCENARIO

Contributor Category	Historic AA		Base Case		Design criteria		Growth Rate		
	Distribution (2001/2002)	**BOD Loading (2001)**	**Per capita AA**	**MM = 1.34xAA**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most Probable Worst Case Scenario**
Residential	77 to 69% (include 4.2% food waste)	9.2 tonne/day	0.053 kg/c/d (1)	0.071 kg/c/d	0.067*+0.01** kg/c/d 0.071[†]+0.01[††] kg/c/d	0.071 kg/c/d (Note 1)	196,765 by year 2021 248,000 by year 2036	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046) 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
C & I	21 to 18% (include 1.5% food waste)	2.44 tonne/day	0.014 kg/c/d (2)	0.02 kg/c/d	0.019 kg/c/d 0.02 kg/c/d	0.02 kg/c/d	196,765 by year 2021 248,000 by year 2036)	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046) 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
Industry	3% unchanged	0.3 tonne/day (AA-2001)	0.002 kg/c/d (AA-2002)	0.003 kg/c/d	0.54 tonne/day (2002 AA x 1.34 PF) 1.11 tonne/day	0.987 tonne/day (Note 2)	196,765 by year 2021 248,000 by year 2036	New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; Existing business at 50% of population growth 1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
Surface Runoff	0 to 10%	0 tonne/day	0.008 kg/c/d	1.9 tonne/day	1.9 tonne/day 1.9 tonne/day	1.9 tonne/day	To remain constant as 10% AA BOD	N/A 0.2% by year 2050, 0.3% from 2051 to 2080	0.2% by year 2050, 0.3% from 2051 to 2080
WW Total	100%	11.9 t/day	0.077	0.1 kg/c/d					

*BOD contribution from Residential sources with complete elimination of food waste discharges to sewer

**10% of total households contribute 0.01 kg/c/d BOD loads by food waste discharges to sewer
[†] BOD contribution from Residential sources (including food waste discharges from existing households)
[††] 80% of new households contribute 0.01 kg/c/d BOD loads by food waste discharges to sewer (accounted for the increase percentage of food grinder usage each year)

Note 1: The proportion of households equipped with food grinders in future years remains the same as in 2002
Note 2: 80% of the difference between lower and upper load parameters

TABLE 4.34A
MAXIMUM MONTHLY (MM) TSS PROJECTIONS AT LGWWTP
FACTORS AFFECTING UPPER AND LOWER BOUNDARIES OF TSS PROJECTIONS UP TO YEAR 2046

	Average Annual (AA)		MML =1.43xAA		Design criteria		Growth Rate		
Contributor Category	**Distribution (2002)**	**TSS Loading (2002)**	**Per capita AA**	**Base Case (Max. Month)**	**Lower & Upper Envelope**	**Remarks**	**Base Case**	**Lower & Upper Envelope**	**Remarks**
Residential	69% (include 5.2% food waste)	10.7 tonne/day	0.061 kg/c/d (1)	0.09 kg/c/d	0.081 + 0.02 kg/c/d	2046: 10% of total households still have garburator (Food Waste assumptions as per DSM memo) (4)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
					0.09 + 0.02 kg/c/d	Existing garburator discharge + Increase of future garburator use (80% of new households) (4)		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
C & I	9% (include 1.9% food waste)	1.32 tonne/day	0.008 kg/c/d (2)	0.01 kg/c/d	0.008 kg/c/d	Food Waste discharge to sewer in 2046: reduced to 10% of 2002 C&I level (4)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
					0.01 kg/c/d	Food waste discharge to sewer remains the same		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
Industry	3% (3)	0.6 tonne/day	0.003 kg/c/d	0.004 kg/c/d	0.858 tonne/day (2002 AA x 1.43 PF)	Assumes user fee policy continues in existing and new business (maybe approaching point of diminishing returns) (3)	In accordance with GVRD population growth (5) (196,765 by year 2021 and 248,000 by year 2036)	New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; Existing business at 50% of population growth (3)	New business: In accordance with population growth developed by Regional Development Division; Improvements in existing business practice
					1.08 tonne/day	Based on 2002 Max. Month record (April) (6)		1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	In accordance with population ranges developed by Regional Development Division (5)
Surface Runoff	19%	2.2 tonne/day	0.016 kg/c/d	4.1 tonne/day	4.1 tonne/day	Loading remains the same	To remain constant as 19% of AA TSS	N/A	
					4.1 tonne/day	Loading remains the same		0.2% by year 2050, 0.3% from 2051 to 2080	Increased precipitation of 10% by 2050; 20% increase by 2080
WW Total	100%	14.8 t/day	0.088	0.13 kg/c/d					

Note:
(1) NSSA Summary 1995-2002
(2) prorated from 2002 flow – C&I Summary (updated)
(3) Memo: Industry Demand Reduction Scenario - NSSA 2002
(4) DSM Scenarios – BOD & TSS Loadings (CP-18-04)
(5) Current GMS 4.0 (Memo: Robert Hicks)
(6) Comments from GVRD at Workshop #1

TABLE 4.34B
MAXIMUM MONTHLY (MM) TSS PROJECTIONS AT LGWWTP
DESIGN CASE SCENARIO

	Average Annual (AA)		MML =1.43xAA		Design criteria		Growth Rate		
Contributor Category	**Distribution (2002)**	**TSS Loading (2002)**	**Per capita AA**	**Base Case (Max. Month)**	**Lower & Upper Envelope**	**Design Case Scenario**	**Base Case**	**Lower & Upper Envelope**	**Most Probable Worst Case Scenario**
Residential	69% (include 5.2% food waste)	10.7 tonne/day	0.061 kg/c/d (1)	0.09 kg/c/d	0.081* + 0.02** kg/c/d	0.09 kg/c/d (Note 1)	196,765 by year 2021 248,000 by year 2036	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
					0.09† + 0.02†† kg/c/d			1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	
C & I	9% (include 1.9% food waste)	1.32 tonne/day	0.008 kg/c/d (2)	0.01 kg/c/d	0.008 kg/c/d	0.01 kg/c/d	196,765 by year 2021 248,000 by year 2036	0.71% (200,000 by year 2021); 0.75% (241,200 by year 2046)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
					0.01 kg/c/d			1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	
Industry	3% (3)	0.6 tonne/day	0.003 kg/c/d	0.004 kg/c/d	0.858 tonne/day (2002 AA x 1.43 PF)	1.04 tonne/day (Note 2)	196,765 by year 2021 248,000 by year 2036	New business at 0.71% growth by year 2021 and 0.75% growth by year 2046; Existing business at 50% of population growth (3)	1.10% (215,189 by year 2021) 0.99% (275,117 by year 2046)
					1.08 tonne/day			1.19% (220,000 by year 2021); 1.04% (283,600 by year 2046)	
Surface Runoff	19%	2.2 tonne/day	0.016 kg/c/d	4.1 tonne/day	4.1 tonne/day	4.1 tonne/day	To remain constant as 19% of AA TSS	N/A	0.2% by year 2050, 0.3% from 2051 to 2080
					4.1 tonne/day			0.2% by year 2050, 0.3% from 2051 to 2080	
WW Total	100%	14.8 t/day	0.088	0.13 kg/c/d					

*TSS contribution from Residential sources with complete elimination of food waste discharges to sewer
**10% of total households contribute 0.02 kg/c/d TSS loads by food waste discharges to sewer
† TSS contribution from Residential sources (including food waste discharges from existing households)
†† 80% of new households contribute 0.02 kg/c/d TSS loads by food waste discharges to sewer

Note 1: The proportion of households equipped with food grinders in future years remains the same as in 2002
Note 2: 80% of the difference between lower and upper load parameters
Note 3: No re-distribution to AIWWTP

FIGURE 4.13
ADWF ENVELOPE BY SECTOR – LGWWTP
(RESIDENTIAL, C&I, INDUSTRY, INFILTRATION)

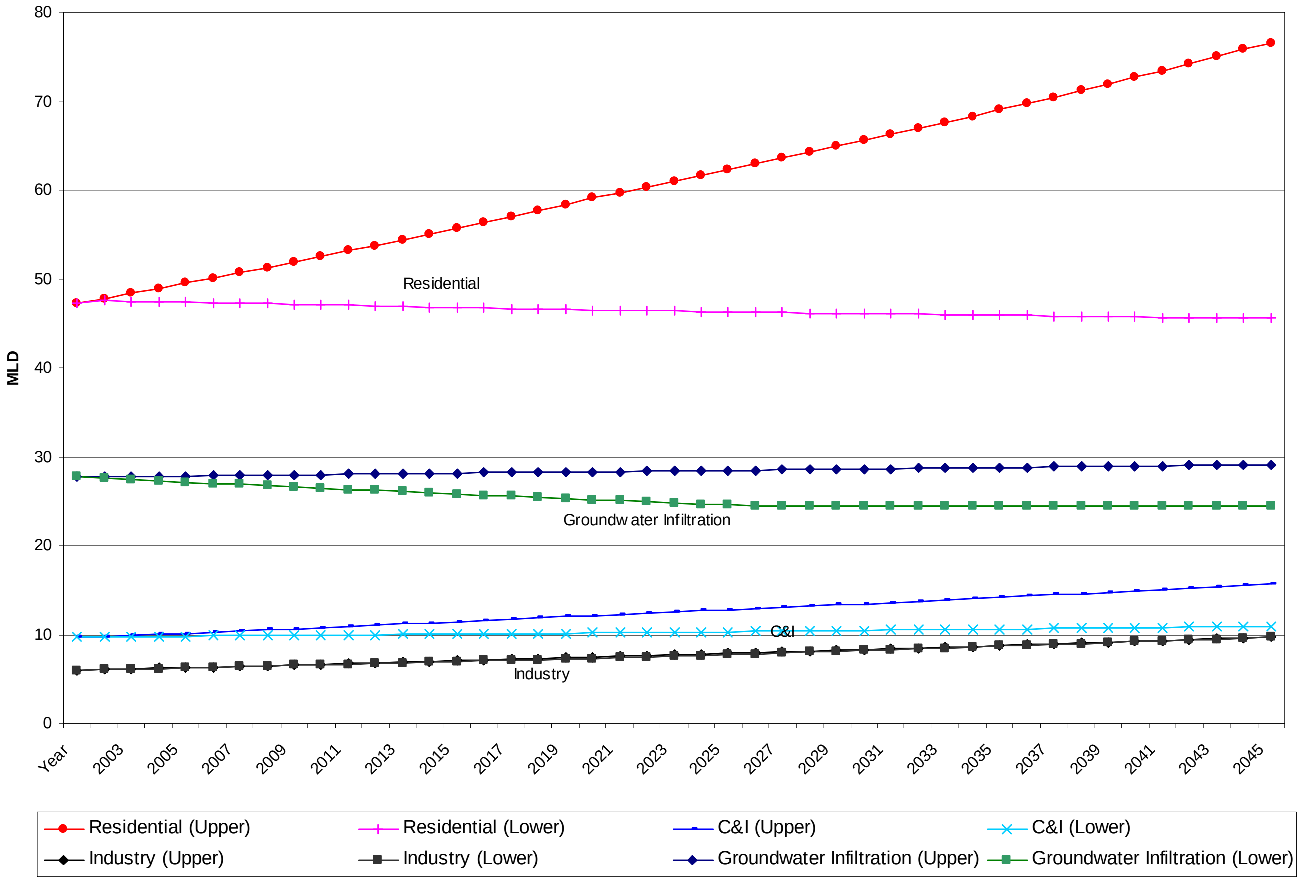


FIGURE 4.14a
LGWWTP UPPER AND LOWER PROJECTION ENVELOPE FOR ADWF (UP TO YEAR 2046)

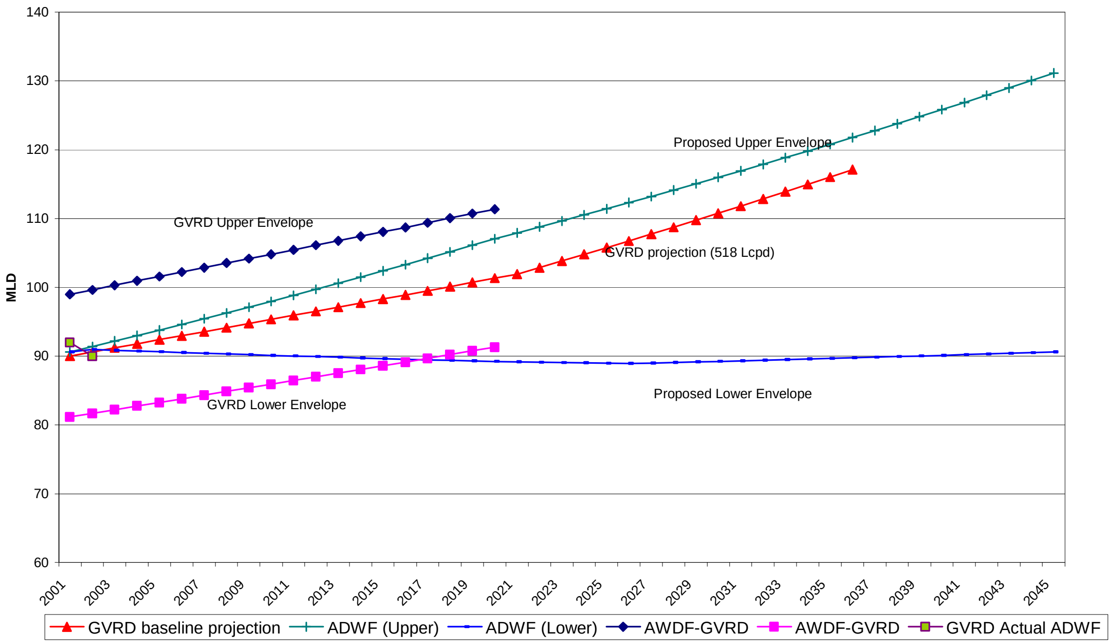


FIGURE 4.14b
LGWWTP DESIGN CASE SCENARIO FOR ADWF

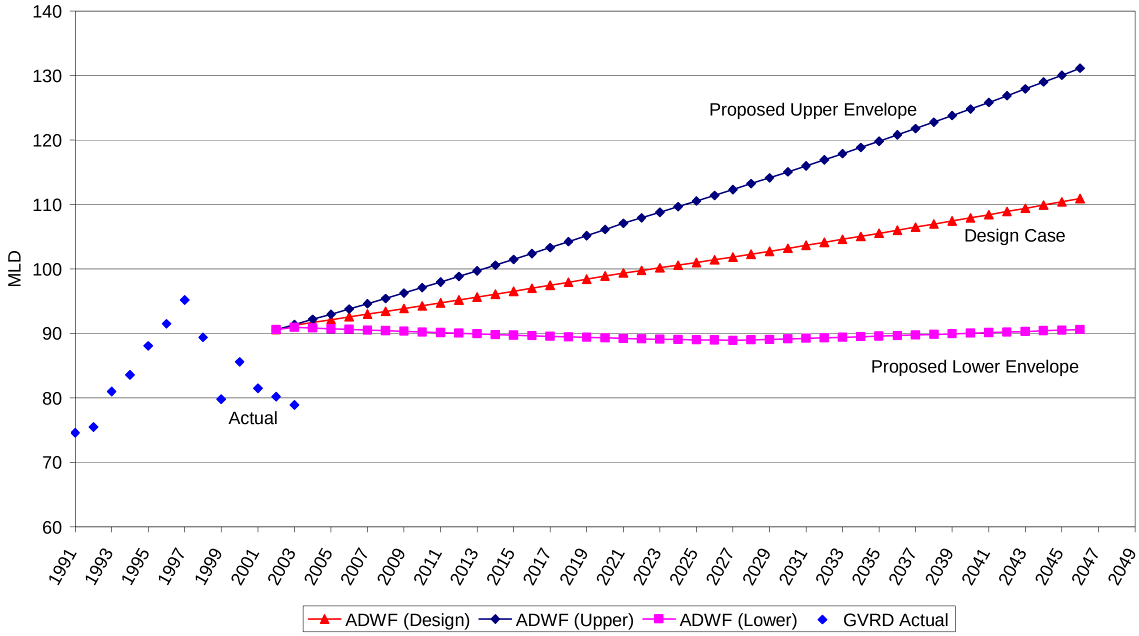


FIGURE 4.15
BOD MAX. MONTH (MM) ENVELOPE PROJECTION BY SECTOR – LGWWTP
(RESIDENTIAL C&I, INDUSTRY)

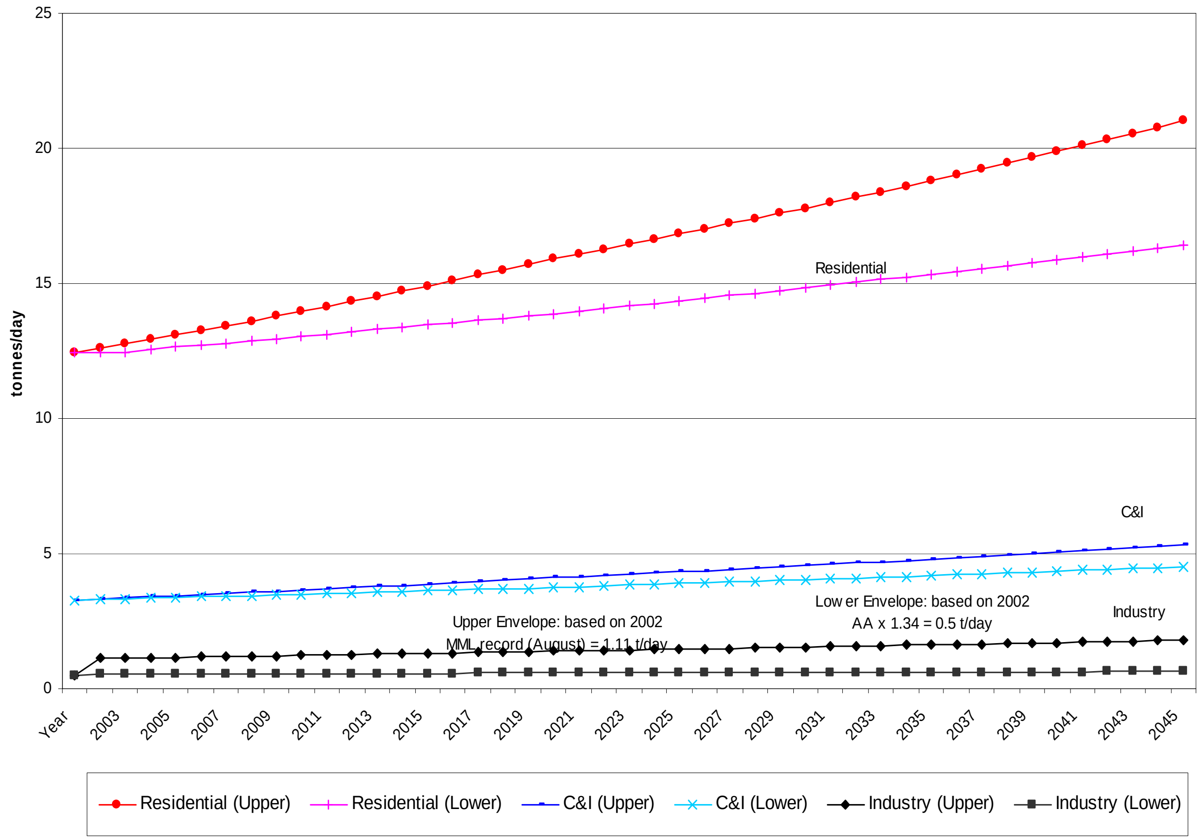


FIGURE 4.16a
MAX. MONTH (MM) BOD UPPER & LOWER PROJECTION ENVELOPE LGWWTP

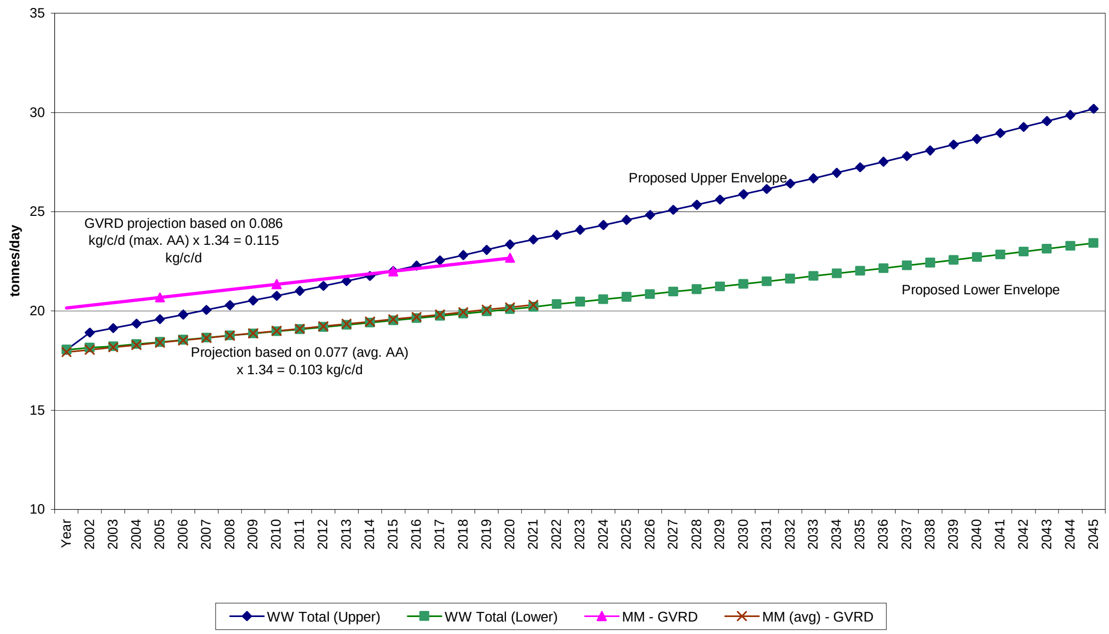


FIGURE 4.16b
DESIGN CASE MAX. MONTH (MM) BOD AT LGWWTP

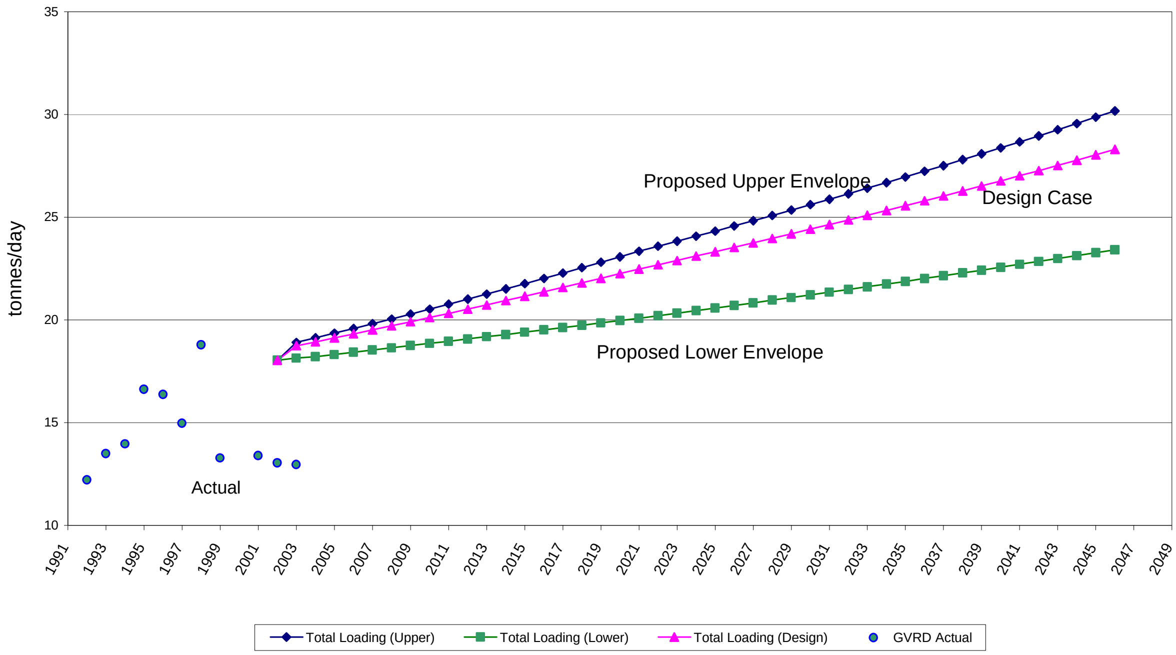


FIGURE 4.17
TSS MAX. MONTH (MM) ENVELOPE PROJECTION BY SECTOR – LGWWTP (RESIDENTIAL C&I, INDUSTRY)

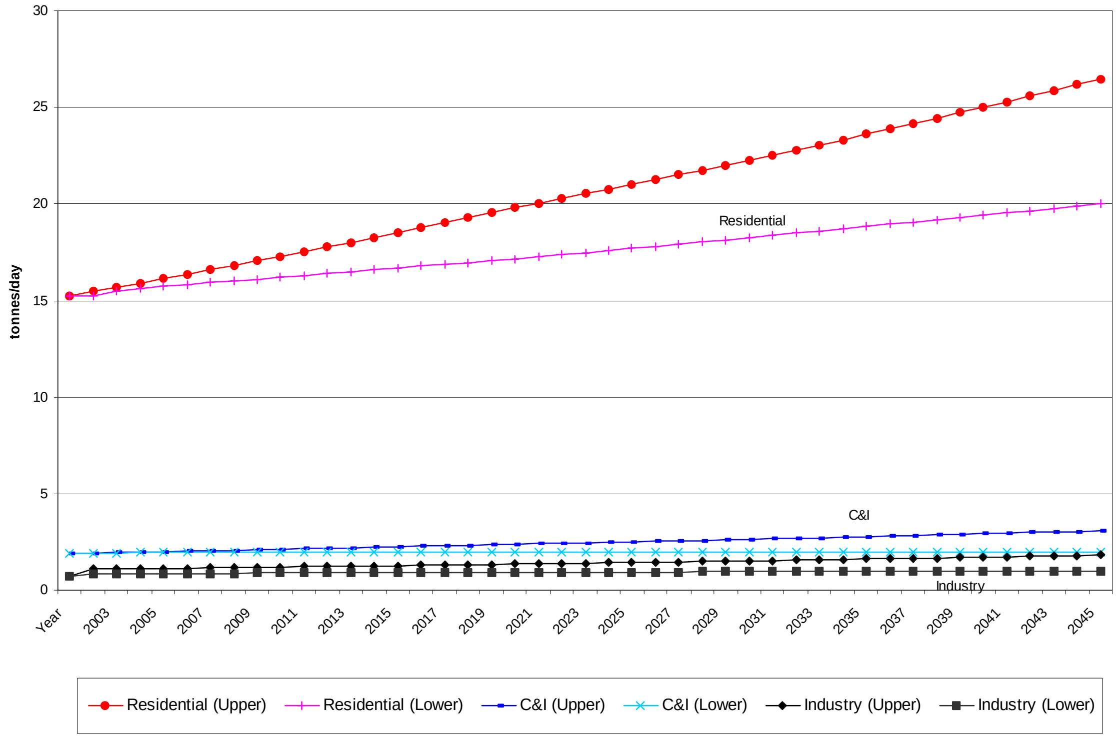


FIGURE 4.18a
MAX. MONTH (MM) TSS UPPER & LOWER PROJECTION ENVELOPES – LGWWTP

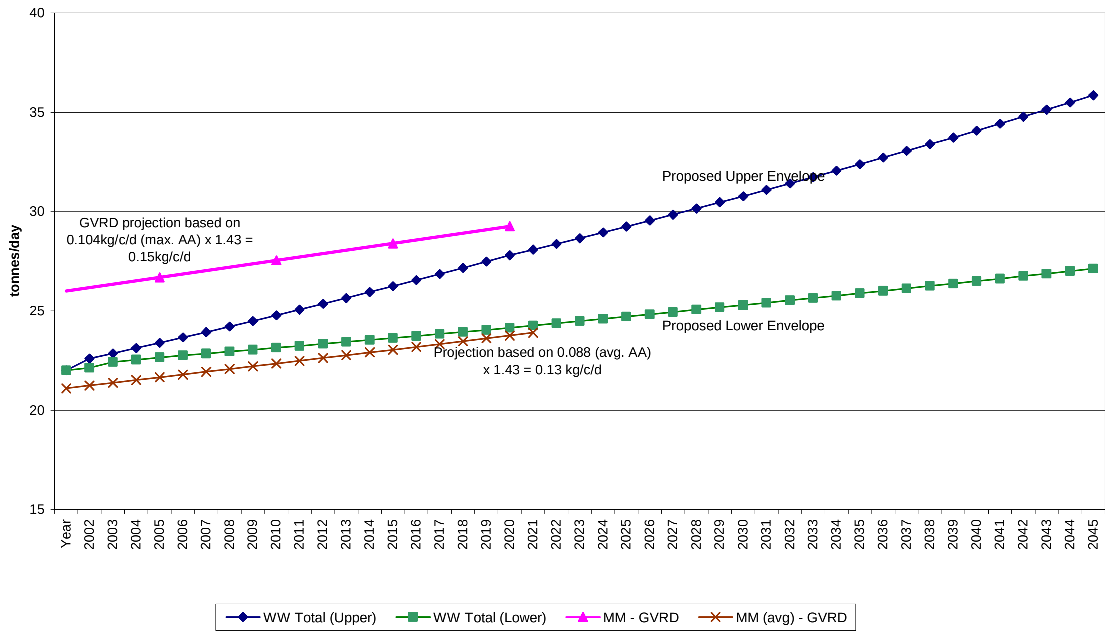
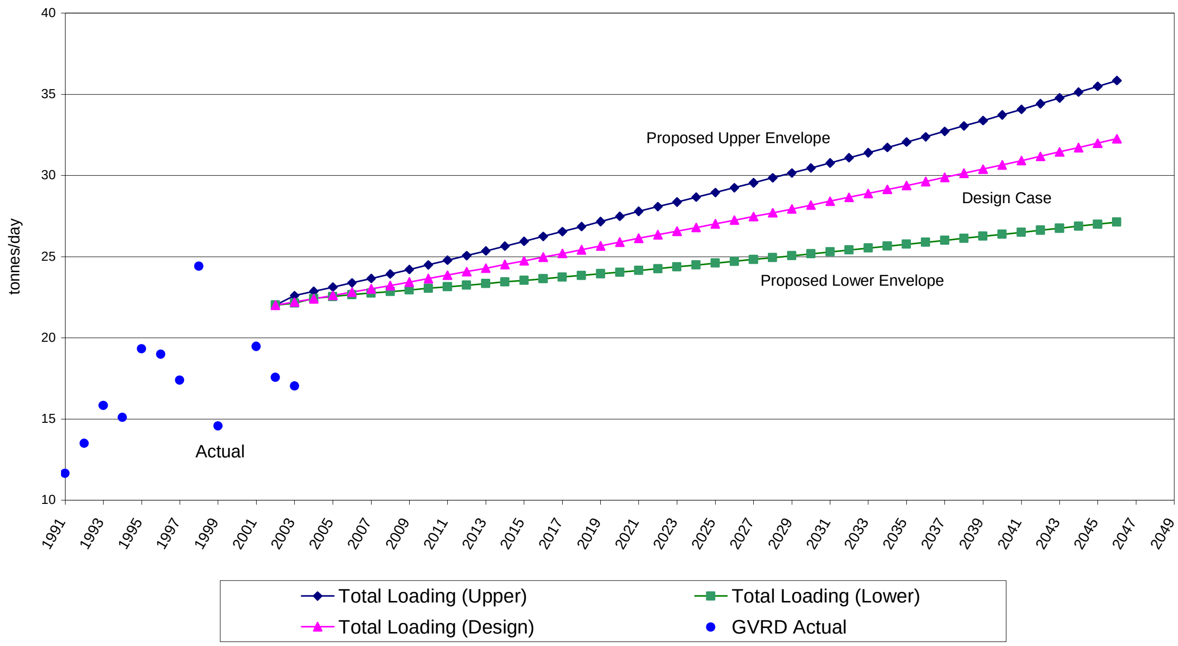


FIGURE 4.18b
DESIGN CASE MAX. MONTH (MM) TSS AT LGWWTP



4.8.3 Flow and Loads Summary

The following Tables 4.35 and 4.36 summarize the projected values of flow and loads for the Iona Island WWTP and the Lions Gate WWTP in year 2021 (design year for interim upgrade), 2036 (design year for IIWWTP build-out to secondary), 2046 (design year for LGWWTP build-out to secondary) and 2081 (design year for Lions Gate sewerage system diversion to Iona Island). Flow and loads from the upper projection envelopes developed in this study exhibit a close similarity to the GVRD baseline projections and are used as design basis.

TABLE 4.35
UPPER PROJECTION ENVELOPE FOR IIWWTP

Year	Flow (ADWF) ML/d	TSS * (t/d)	BOD * (t/d)
2002 (base year)	436	93	102
2011	466	102	112
2021	502	112	124
2036	516	117	128

*Contribution from Trucked Liquid Wastes not included

TABLE 4.36
UPPER PROJECTION ENVELOPE FOR LGWWTP

Year	Flow (ADWF) ML/d	TSS (t/d)	BOD (t/d)
2002 (base year)	91	22	18
2011	98	25	21
2021	107	28	23
2031	116	31	26
2046	131	36	30
2081	150	42	36

Figure 4.19 presents the upper ADWF projection envelopes up to year 2101 for IIWWTP, LGWWTP and their combined flow for both WWTPs in comparison to the GVRD projections based on the per capita ADWF of 704 L/c/d and 518 L/c/d for IIWWTP and LGWWTP respectively. The ADWF projected contributions from Industry and Ground Infiltration sources for IIWWTP beyond year 2036 and for LGWWTP beyond year 2046 are assumed to remain constant.

Most probable base case scenario developed in this study will be used for the final options. Similar to Tables 4.35 and 4.36, Tables 4.37 and 4.38 summarize the flow and loads for Iona Island WWTP and Lions Gate WWTP respectively.

TABLE 4.37
DESIGN CASE FOR IIWWTP

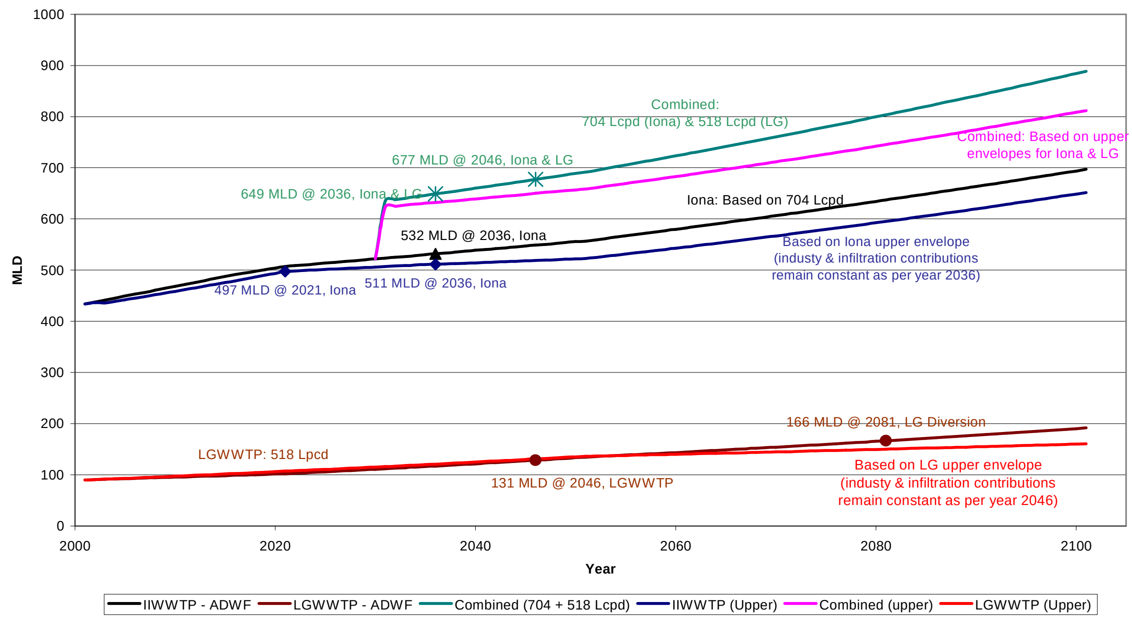
Year	Flow (ADWF) ML/d	TSS* (t/d)	BOD* (t/d)
2002 (base year)	436	99	104
2011	445	108	114
2021	456	116	124
2036	441	119	127

*: Contribution from trucked liquid wastes not included.

TABLE 4.38
DESIGN CASE FOR LGWWTP

Year	Flow (ADWF) ML/d	TSS* (t/d)	BOD* (t/d)
2002 (base year)	91	22	18
2011	95	24	20
2021	99	26	22
2036	106	30	26
2046	111	32	28
2081	116	39	34

FIGURE 4.19
AVERAGE DRY WEATHER FLOW (ADWF) TO YEAR 2101
IONA ISLAND & LIONS GATE WWTPS



5 DESCRIPTION OF EXISTING PLANT

5.1 IONA ISLAND WWTP

5.1.1 General

Iona Island WWTP (IIWWTP) is a primary treatment facility that provides treatment of combined storm and sanitary sewers from the Vancouver Sewerage Area (VSA). The plant was commissioned in 1963 and has undergone major expansions in 1972, 1978, 1985 and 1986. The plant currently serves a population of approximately 620,000 people in the City of Vancouver, the University Endowment Lands, part of Richmond and Burnaby. IIWWTP also receives both domestic and non-domestic trucked liquid waste (TLW) generated in the region. The domestic TLW is combined with the raw sewage influent and treated by the main plant processes. The non-domestic TLW is pre-treated by screening and settling prior to entering the primary treatment units in the main system. Descriptions of the TLW treatment facility are included in Appendix 1.

5.1.2 Process Description

The plant consists of a series of process units, including preliminary treatment, primary treatment, and sludge handling. The process schematics and plant layout are illustrated in Figures 5.1 and 5.2 respectively. The design values and capacities of preliminary treatment, primary treatment, and sludge handling are summarized in Tables 5.1, 5.2 and 5.3, respectively.

5.1.2.1 Preliminary Treatment

The preliminary treatment process consists of bar screens, influent pumps, grit chambers, pre-aeration tanks, and flow distribution channels.

Bar Screen

The joint flows of three 1.68 m diameter influent siphons across the Fraser River, a 0.96 m diameter sewer from airport, domestic TLW, and sludge thickener supernatant recycle, enter the plant headworks area at the bar screens. Six vertical bar screens with openings of 12.7 mm are operated to remove coarse solids from the raw influent. Captured screenings are collected and transferred by mechanical rakes to the compactors and hoppers. The annual screenings productions are recorded at about 370, 360, and 270 tonnes/year (by wet weight) in 2000, 2001 and 2002, respectively. The bar screens are currently undergoing a modification for mechanical feature upgrade.

Influent Pump

After the bar screens, wastewater is collected in the influent pump wet well and then pumped to each designated grit chamber by six centrifugal pumps. The rated full pump capacity is about 23.2 m^3/s (four 4.8 m^3/s and two 2.0 m^3/s, 2,006 ML/d in total). The plant internal recycle flows are also collected in the influent pump wet well, including screening return, plant drain, and thickener supernatant recycle. The influent pumps are equipped with variable frequency drive (VFD) for the flow rate control, and the pumps are operated alternatively during low flow conditions.

Grit Chamber

The grit chamber consists of four large channels and two small channels. The channels are long and narrow allowing the grit to settle by gravity. Longitudinal scrapers at the channel bottom are used to collect grit to the sumps located at the influent end of the channels. Collected grit is pumped to one of the two grit cyclones and classifiers where the grit is concentrated and classified or separated before disposal in the grit hoppers. The annual grit productions are recorded about 1,720 and 1,610 tonnes/year (by wet weight) in 2001 and 2002, respectively. The reject overflow from the grit cyclones are returned to the grit chambers. The overflow from the grit chamber exits through flow proportional weirs to the common flow distribution channel.

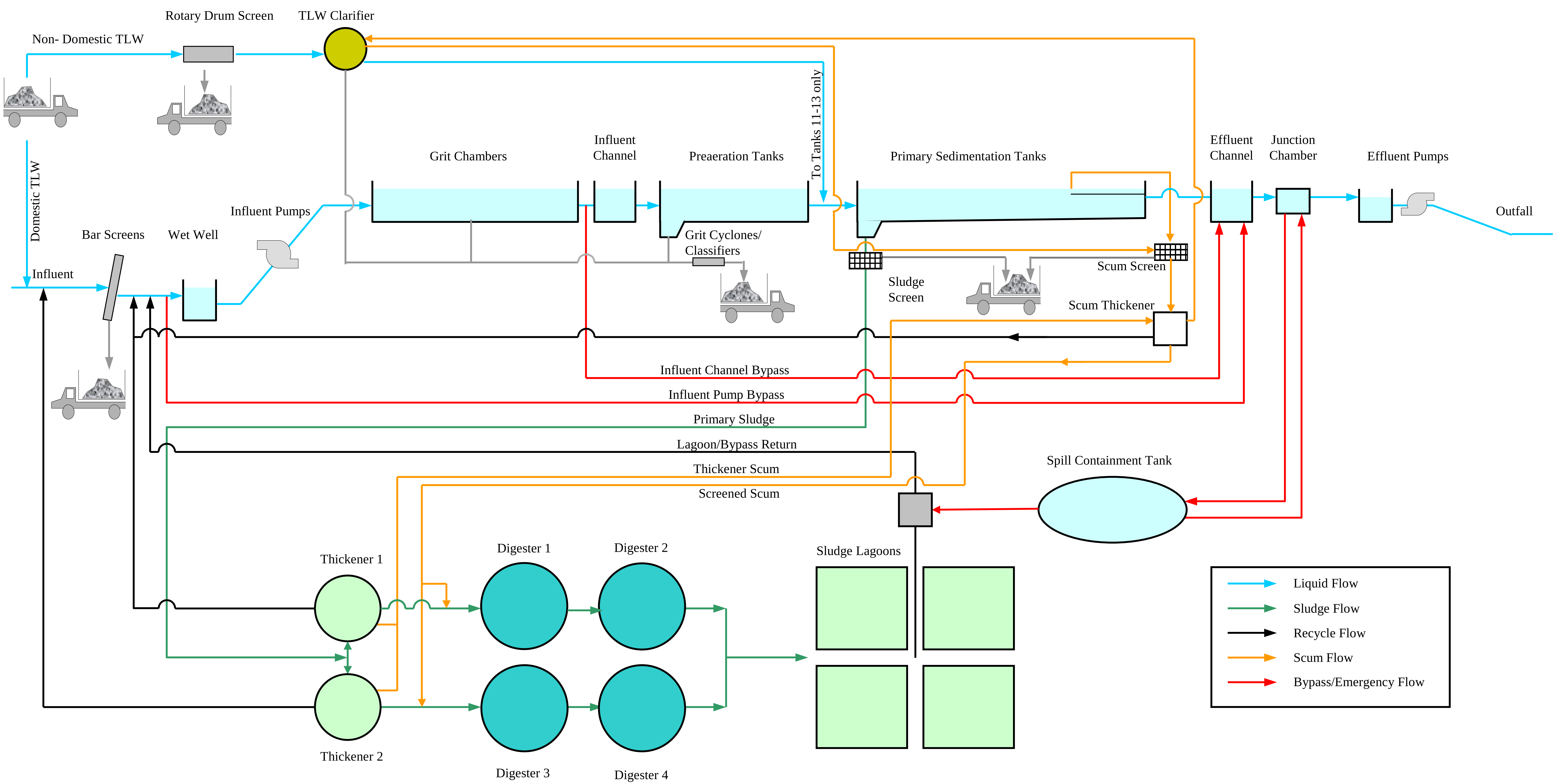
Flow Distribution Channel

The grit chamber effluents are collected in the common flow distribution channel and split toward the north and south distribution channels. The flow is further distributed through the individual slider gate into each per-aeration tank.

Pre-aeration Tanks

The flow in the distribution channels is split into fifteen rectangular pre-aeration tanks, thirteen smaller tanks (tanks 1-13) of the same size and two larger ones (tanks 14 and 15). From the distribution channels, flow is distributed by individual slide gates to the north pre-aeration tanks (tanks 6-13) and south pre-aeration tanks (tanks 1-5,14 and 15). Tanks 11-13 also receive non-domestic TLW from the TLW pretreatment facility. Air is supplied to the tanks by five blowers through the air diffusers located at the bottom of each tank. Aeration helps to suspend small biological solids and settle heavier solids (grit). The dissolved oxygen concentration (DO) is also increased to prevent septicity. Grit is collected by screw conveyors located on either side of each tank to the grit hoppers where it is pumped to the grit cyclones and classifiers for dewatering. Reject flows from the cyclones and classifiers are discharged back to the influent channel near pre-aeration tank 6.

FIGURE 5.1 IONA ISLAND WWTP PROCESS SCHEMATICS



Digester 1 currently out of service (2004)

FIGURE 5.2 IONA ISLAND WWTP LAYOUT

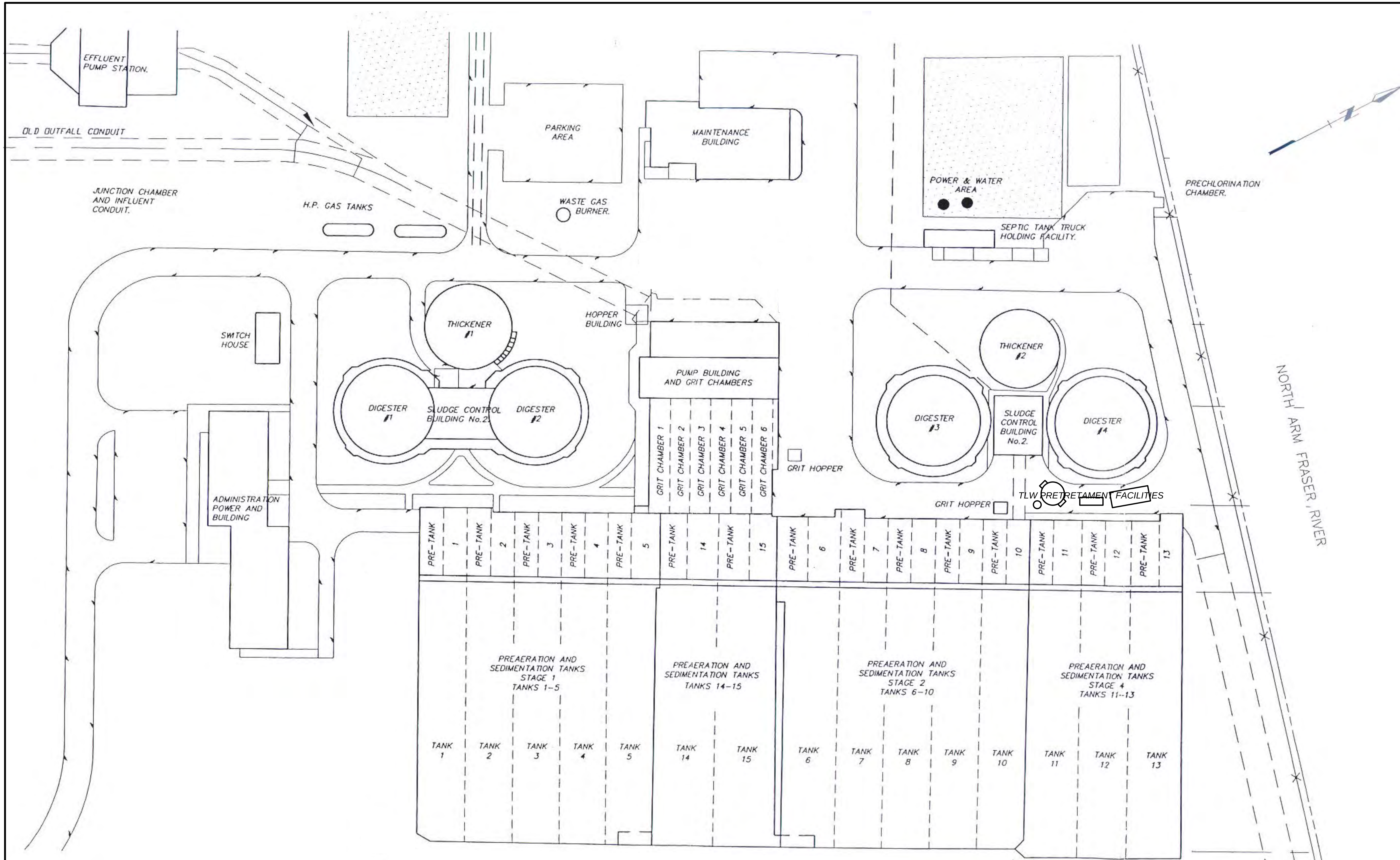


TABLE 5.1
IIWWTP PRELIMINARY TREATMENT PROCESS UNIT CAPACITY AND DESIGN VALUES

Bar Screen	Number	6			
	Capacity per Screen	363	MLD		
	Total Capacity	2152	MLD		
	Maximum Flow Velocity	4.5	m/s		
	Rake Speed	0.17	m/s		
	Screening Compactor				
	Number	2			
	Total capacity	1.7	m^3/hr		
Influent Pump		Large		Small	
	Number	4		2	
	Capacity per Pump	354	MLD	220	MLD
	Total Capacity	1856	MLD		
Grit Chamber		Large		Small	
	Number of channels	4		2	
	Maximum Flow	429	MLD	183	MLD
	Velocity at maximum flow	0.37	m/s	0.37	m/s
	Total Capacity	2081	MLD		
	Grit Removal Capacity	0.012	m^3 / 1000 m^3 flow		
	Grit Hopper	Number		2	
		Storage Capacity per hopper		16	m^3
	Grit Cyclone and Classifier	Number		5	
		Capacity per Cycle/Classifier		16	L/s @ 8 psig
	Grit Pump	Number		7	
		Capacity per Pump		8.87	L/s @ 6.71 m TDH
Flow Distribution Channel		North		South	
	Number	1		1	
	Width	2.4	m	2.4	m
	Depth	3.15	m	3.15	m
	Length	68	m	68	m
Pre-aeration Tank		Large		Small	
	Number	2		13	
	Surface Area	195	m^2	159	m^2
	Average Depth	4.57	m	4.57	m
	Air Supplied per Tank	14.2	m^3/min	14.2	m^3/min
	ADWF Detention Time	0.64	hours	0.67	hours
	PWWF Detention Time	0.16	hours	0.17	hours
	Maximum Capacity per Tank	127	MLD	98	MLD
	Total Maximum Capacity	1523	MLD		

5.1.2.2 Primary Treatment

The primary treatment consists of the primary sedimentation tanks, primary effluent channel and effluent pumps.

Primary Sedimentation Tanks

Flows from the pre-aeration tanks are introduced to the primary sedimentation tanks through series of orifices. The low velocities flow in the sedimentation tanks allows suspended solids to accumulate on the bottom by gravity and scum to rise to the surface. The settled solids, or sludge, are scraped by chain and flight to the sumps near tank entrance and pumped to the sludge gravity thickeners. Scum is collected in scum sumps then pumped to the scum screen. After the scum is screened, it is either diverted to the TLW sedimentation tank or to the scum thickener and digester for further treatment. The primary effluent is collected by launders and conveyed by the north and south effluent channels to the effluent pump wet well.

Effluent Channel / Effluent Pumps / Outfalls

Two Parshall flumes with throat width of 7' are installed in each of the northern and southern effluent channel. Six centrifugal effluent pumps are located in the wet well to convey the effluent through outfall for discharge. The rated full pump capacity is about 22.2 m^3/s (3.7 m^3/s each, 1,920 ML/d in total). The effluent is discharged to the Georgia Strait via two 7.7-km deep sea outfalls.

TABLE 5.2
IIWWTP PRIMARY TREATMENT PROCESS UNIT CAPACITY AND DESIGN VALUES

		Large (Tank 14 ~ 15)	Small (Tank 1 ~ 13)
Primary Sedimentation Tank	Number	2	13
	Surface Area	976 m^2	759 m^2
	Average Depth	2.74 m	2.74 m
	ADWF Detention Time	1.92 hours	1.93 hours
	ADWF SOR* @ 497 MLD	34.2 m^3/m^2d	34.2 m^3/m^2d
	PWWF SOR* @ 1530 MLD	130 m^3/m^2d	130 m^3/m^2d
	ADWF Capacity per Tank	34 MLD	26 MLD
	Maximum Capacity per Tank	127 MLD	98 MLD
	Total ADWF Capacity	404 MLD	
	Total Maximum Capacity	1523 MLD	
	Total effective weir length	2688 m	
	ADWF WOR**	150 m^3/m^2d	
	PWWF WOR**	567 m^3/m^2d	
	SOR*: Surface Overflow Rate		
	WOR**: Weir Overflow Rate		
Effluent Pump	Number	6	
	Capacity per Pump	320 MLD @ 21.3m TDH	
	Total Capacity	1920 MLD	

5.1.2.3 Solids Handling

The sludge handling consists of the gravity thickeners, digesters and storage lagoons.

Sludge Thickener

Two gravity thickeners, both operational, are operated to receive screened sludge from the primary sedimentation tanks. Each thickener, circular in shape, consists of an influent well and a rotating sludge collector. Solids settle to the bottom by gravity and are thickened by the settling and compaction of solids. The thickened sludge is pumped to the digesters via heat exchange loops. Scum is collected on the top and sent to the scum thickener.

Anaerobic Digester

The plant has four anaerobic digesters, with digesters # 1 and # 2 and digesters # 3 and # 4 operating in series. Thickened sludge from the thickeners and thickened scum from the scum thickeners are stabilized in the digesters and undergo mesophilic digestion at about $37^{o}C$. The digesters are mixed by gas lances contained within a central draft tube. Gas mixing is supplemented by recirculation pumps. Off-gas produced is utilized in the cogeneration engines, flared off as waste gas, or recycled back to the digesters to facilitate circulation of sludge. The design gas production was estimated about 55 m^3/kg VSS reduced, producing 1,700 kW of energy at 5,500 kJ/kWh.

Lagoon

Digested sludge, or biosolids, from the digesters is pumped to one of the four lagoons adjacent to the plant. Biosolids settle and accumulate on the bottom. The liquid level in the lagoons is maintained constant by circulating liquid among the lagoons or by pumping supernatant back to the influent pump wet well. When the lagoons reach the storage capacity, bottom sludge is removed by trucks to the east side of the plant for on-site stockpiling.

TABLE 5.3
IIWWTP SLUDGE HANDLING PROCESS UNIT CAPACITY AND DESIGN VALUES

Sludge Thickener	Number	2			
	Diamater	19.81	m		
	Side Water depth	3.05	m		
	Solids Loading	45.5	kg SS/m^2/d		
	Overflow Rate	31.8	m^3/m^2d		
	Raw Sludge Concentration	~0.15%			
	Thickened Sludge Concentration	~6%			
	Raw Sludge Pump	Number		9	
		Capacity per Pump		37.89	L/s @ 11.58 TDH
Anaerobic Digester	Number	4			
	Diamater	24.38	m		
	Side Water depth	10.67	m		
	Solids Loading	96.1	kg/m^3/30-day month		
	Hydraulic Retention Time (HRT)	20	days @ 6% sludge solids		
	Average Temperature	37	^oC		
	Circulation Pumps	8			
	Capacity per Circulation Pump	37.89	L/s @ 16.76 TDH		
Lgaoon	Number	4			
	Total Volume	334,180	m^3		

5.1.3 Current Facility Capacity

The plant design flows and loads are summarized in Table 5.4, as well as the 2002 average conditions are also listed for comparison. The capacity of the various unit processes as described in this section where obtained from the report titled "IIWWTP Process Characterization, GVRD, 2001".

A forecast of the plant effluent quality based on the increase in average dry weather flow to 2020 is included in Appendix 10 together with a discussion on factors to be considered when establishing the capacity of primary sedimentation tanks. The forecast of effluent quality from 2004 to 2020 will assist in establishing a time line for the interim upgrades.

5.1.3.1 Liquid Stream

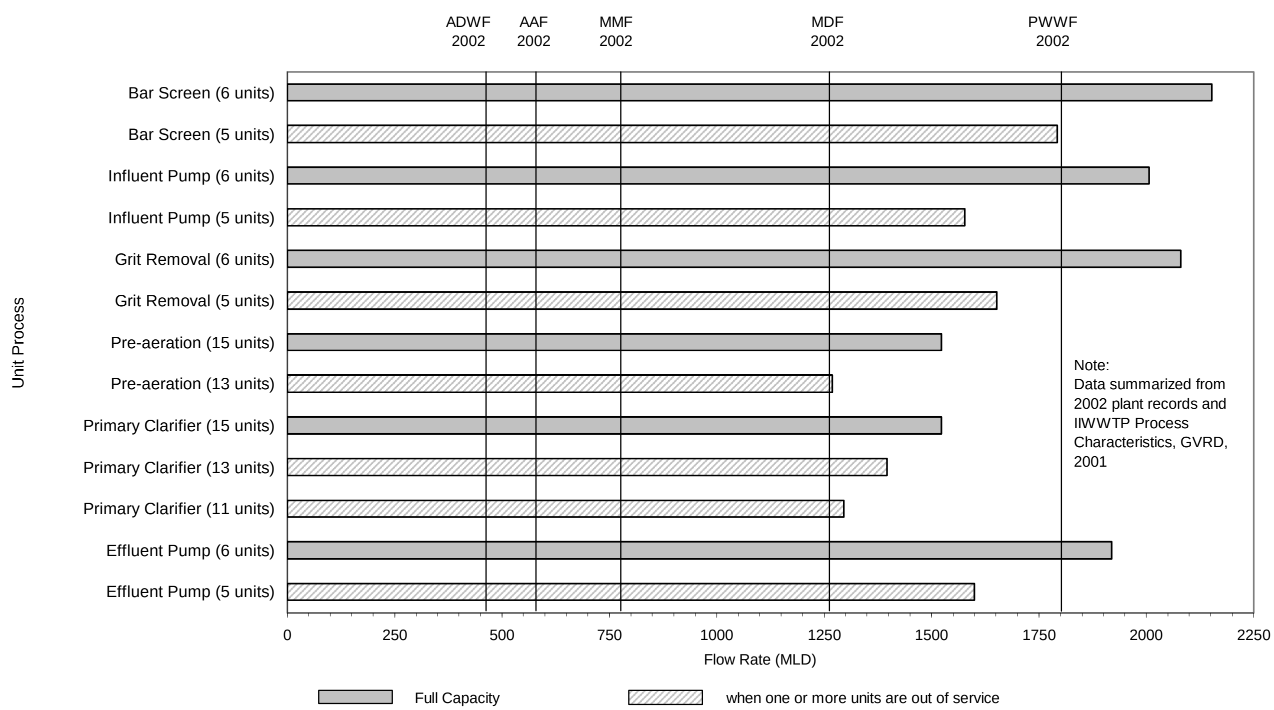
The flow conditions of 2002 averages, including average dry weather flow (ADWF), average annual flow (AAF), maximum month flow (MMF), maximum day flow (MDF) and peak wet weather flow (PWWF), are shown in Figure 5.3 against the process capacity of the liquid treatment processes. As indicated above the process capacity is based on the 2001 GVRD report. The instantaneous PWWF in 2002 had exceeded the pre-aeration and primary sedimentation capacities. However, the maximum daily flow is still below the design value of 1530 ML/d.

TABLE 5.4
IIWWTP PLANT DESIGN FLOWS/LOADS AND 2002 AVERAGES

Parameter	Unit	Design value*		2002 annual average	
		Total	Per capita	Total	Per capita
Population	person	640,000	-	621793	-
ADWF	MLD	497	777 L/c/d	459	738 L/c/d
AAF	MLD	575	-	574	-
MMF	MLD	-	-	781	-
MDF	MLD	-	-	1,263	-
PWWF	MLD	1,530	-	1,806	-
BOD	mg/L	200	-	128	-
BOD	kg/d	80,600	0.126 kg/c/d	68,579	0.110 kg/c/d
BOD removal	%	35	-	33	-
TSS	mg/L	250	-	130	-
TSS	kg/d	100,750	0.157 kg/c/d	69,509	0.112 kg/c/d
TSS removal	%	60	-	59	-

*: IIWWTP Process Characterization, GVRD, 2001

FIGURE 5.3
IIWWTP EXISTING PROCESS UNIT CAPACITY - LIQUID STREAM



Since solids removal is the main design function for the primary treatment plant such as IIWWTP, the surface overflow rate (SOR) and weir (launder) overflow rate (WOR) are crucial for the plant operation. Other factors which will affect the performance of primary sedimentation tanks include detention time, wastewater characteristics such as freshness, particle types and water temperature. The SOR and WOR for 2002 at ADWF and PWWF conditions are calculated in Table 5.5. In comparisons with the plant design and typical design values, the 2002 conditions have exceeded or are in the high end of the design ranges. The treatment capacity of primary sedimentation seems to be the bottleneck of the system.

Other configuration limitations and operational factors may also have significantly affected the system performance, including the followings:

- Flow distribution,
- Number of influent pumps in service,
- Trucked liquid waste contribution,
- Influent wastewater characteristics (e.g. soluble BOD and solids settleability),
- Bottom sludge withdraw capacity.

These surface flow rates were calculated based on the assumptions of even flow distribution among the sedimentation tanks and all 15 tanks were online in service. Nevertheless, proper flow distribution has always been a constraint at IIWWTP, and one or two sedimentation tanks were usually offline for maintenance. Combinations of these factors should be considered in accordance with future retrofit and process control. Mitigations of these limiting factors are crucial to the plant operation, especially when the plant is to be operated to treat the maximum process capacity.

TABLE 5.5
SURFACE OVERFLOW RATE AND WEIR OVER FLOW RATE OF IIWWTP PRIMARY SEDIMENTATION

Parameters	Units	Design	2002 Averages	Typical Design Range
SOR at ADWF	$m^3/m^2/d$	34	38	32 ~ 48
SOR at PWWF	$m^3/m^2/d$	130	153	80 ~ 120
WOR at ADWF	$m^3/m/d$	150	170	124 ~ 496
WOR at PWWF	$m^3/m/d$	567	670	

SOR: surface overflow rate
WOR: weir overflow rate

5.1.3.2 Solids Stream

The capacities of solids handling in the thickeners and digesters are identified in Figure 5.4 and Figure 5.5, respectively. The loading rates for 2002 averages are also listed in Table 5.6, in comparisons with the plant design values and typical design criteria.

FIGURE 5.4
IIWWTP EXISTING PROCESS UNIT CAPACITY – SLUDGE THICKENER

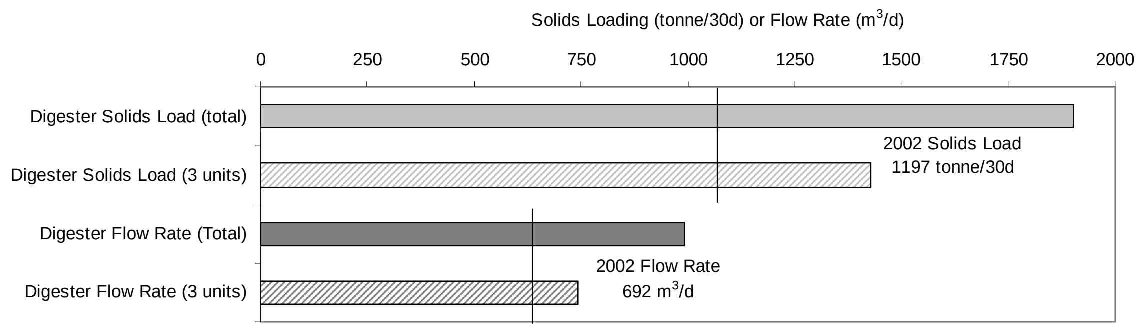


FIGURE 5.5
IIWWTP EXISTING PROCESS UNIT CAPACITY – SLUDGE DIGESTER

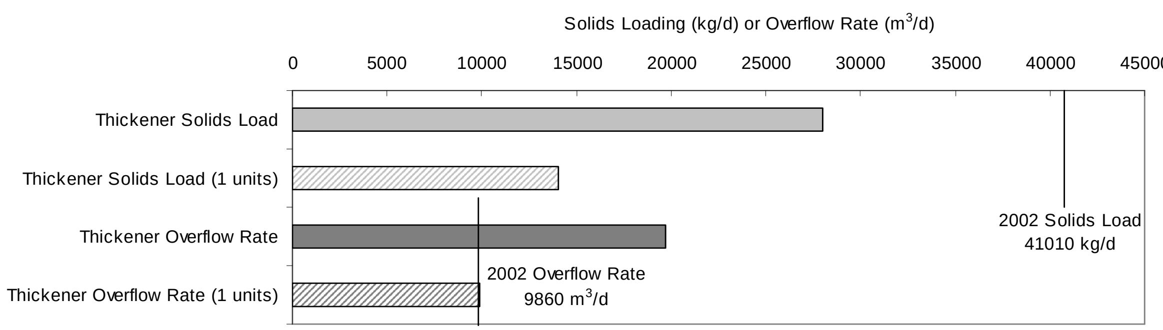


TABLE 5.6
GRAVITY THICKENER AND DIGESTER SOLIDS AND HYDRAULIC LOADS

Parameters	Units	Design	2002 Averages	Typical Design Range
Thickener Solids Load	$kg/m^2/d$	45.5	67	87 ~ 136
Thickener Overflow Rate	$m^3/m^2/d$	32	16	-
Digester Solids Load	$kg/m^3/30d$	96	80/105*	72 ~ 130
Digester Retention Time	d	20 @ 6% solids	28/21* @ 5.7% solids	10 ~20**

*: with three digesters only
**: based on high-rate operation

These solids and hydraulic loads were found to be within the plant design and typical design criteria, except the thickener solids load. The design load as indicated in a GVRD report is 45.5 $kg/m^2/d$, which is significantly lower than the typical design range of 87 ~ 136 $kg/m^2/d$. The current average solid load at the thickeners with both units in service was about 67 $kg/m^2/d$, which indicates that the existing units can handle higher solids load than the design values and achieve approximately 97% of solids capture.

The 97% solids capture has been estimated by a mass balance of solids removed from the sedimentation tanks and solids load entering the digesters. Plant staff has reported some difficulties in operating the gravity thickener to achieve consistent efficiency. Intensive operator attention is needed in handling the load variances and floating debris, primarily due to the nature of sludge withdraw method/pattern and varied sludge concentrations.

In fact, the gravity thickener performance is dependent on many factors, which include the primary sludge characteristics, sludge pump schedule (intermittent or continuous), PST scraper efficiency, and sludge collection hopper configurations etc. Currently, the primary sludge withdraw is based on torque control to determine the cycle and duration. The numbers of scraper blade and different hopper configuration may have resulted in the inconsistency and difficulty of the gravity thickener operation. The implementation of chemically enhanced primary (CEP) treatment will also increase the operational difficulty in the gravity thickener (e.g. solids loading and supernatant quality). It is recommended to rerate the gravity thickener capacity, in conjunction with any operational upgrade (e.g. sludge withdraw control) and loading increases (e.g. future CEP operation).

5.1.4 Effluent Quality

5.1.4.1 Liquid Stream

In accordance with the permit requirements, the effluent quality are presented as daily total effluent flow, daily BOD and TSS concentrations, daily BOD and TSS loads, from January 1997 to December 2002. The daily total effluent flows are shown in Figure 5.6, with a maximum flow of 1,375 ML/d recorded in March 1997. The effluent BOD concentrations and loads are shown in Figure 5.7 and 5.8, respectively. In 2002, one instance of BOD concentration and load non-compliance was recorded in March 2002. The effluent TSS concentrations and loads are shown in Figure 5.9 and 5.10, respectively. No violation of the TSS concentration and load was reported in 2002. The effluent toxicity LC_{50} test results are summarized in Table 5.7, and the effluent samples of June and August of 2002 failed to meet 100% LC_{50} requirement.

FIGURE 5.6
IIWWTP DAILY TOTAL EFFLUENT FLOW (1997 ~ 2002)

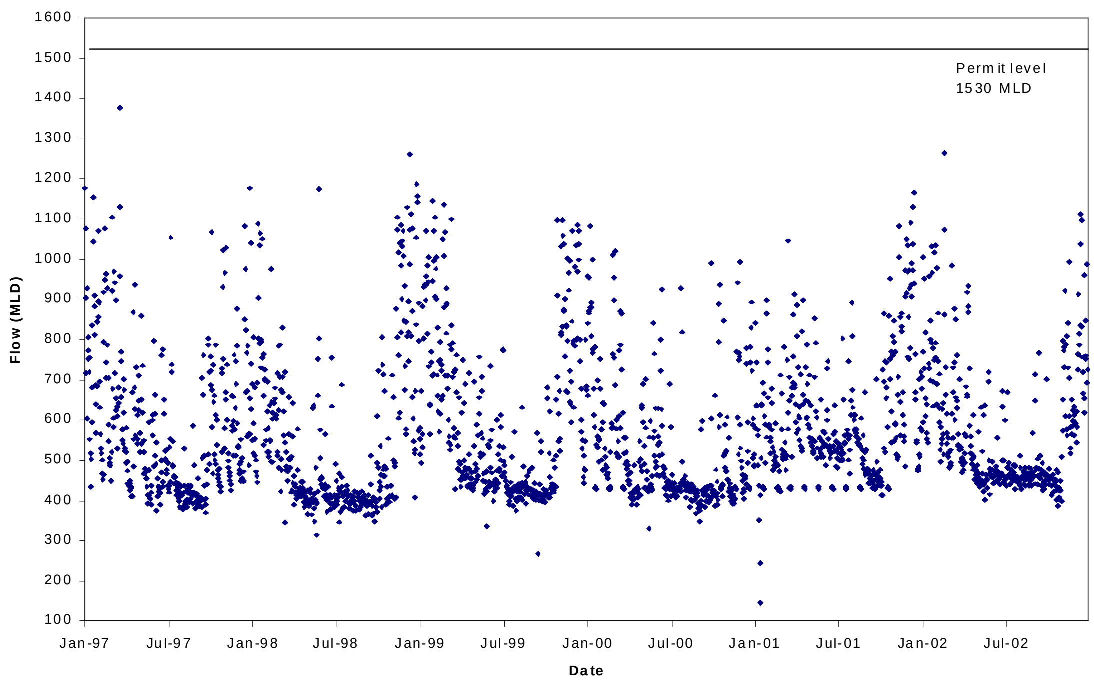


FIGURE 5.7
IIWWTP EFFLUENT BOD CONCENTRATION (1997 ~ 2002)

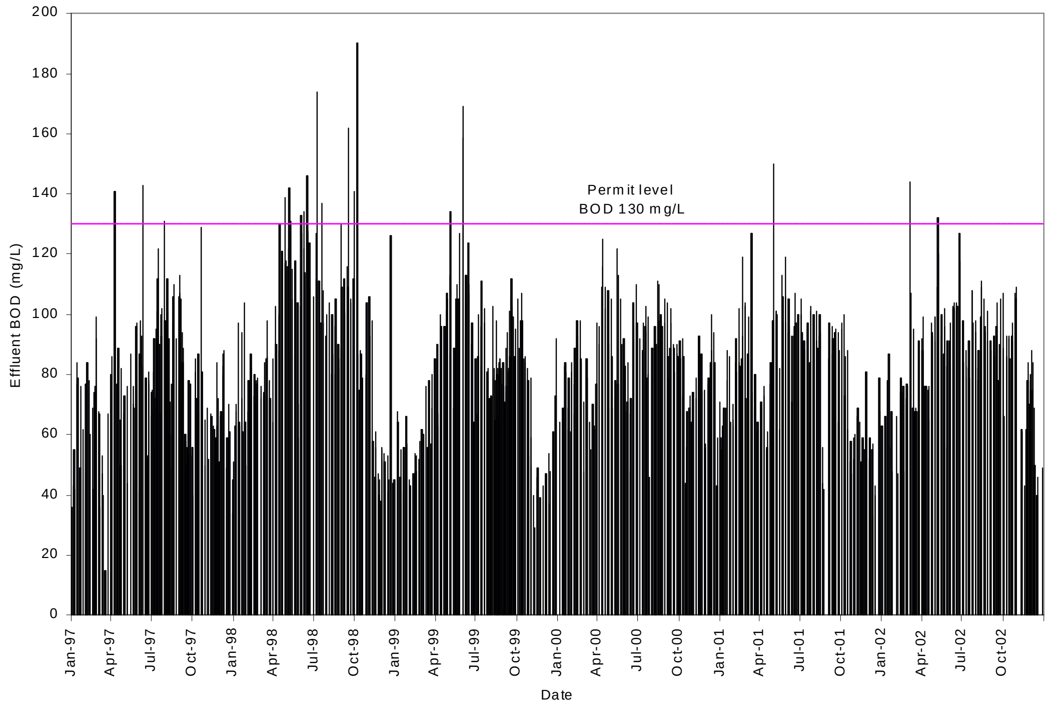


FIGURE 5.8
IIWWTP EFFLUENT BOD LOAD (1997 ~ 2002)

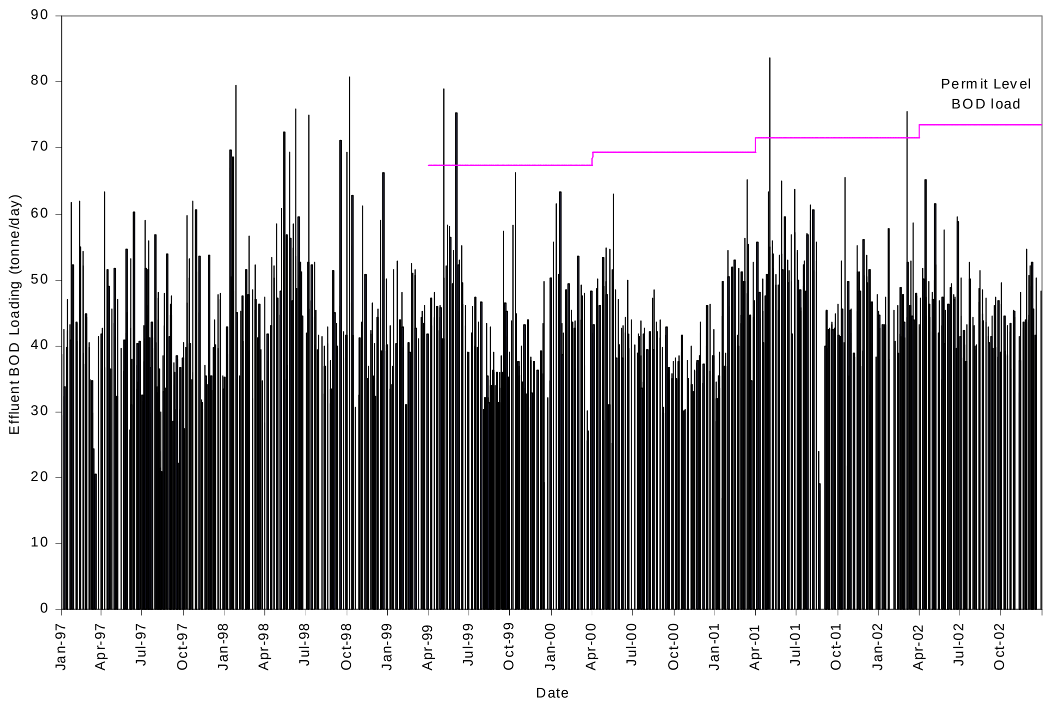


FIGURE 5.9
IIWWTP EFFLUENT TSS CONCENTRATION (1997 ~ 2002)

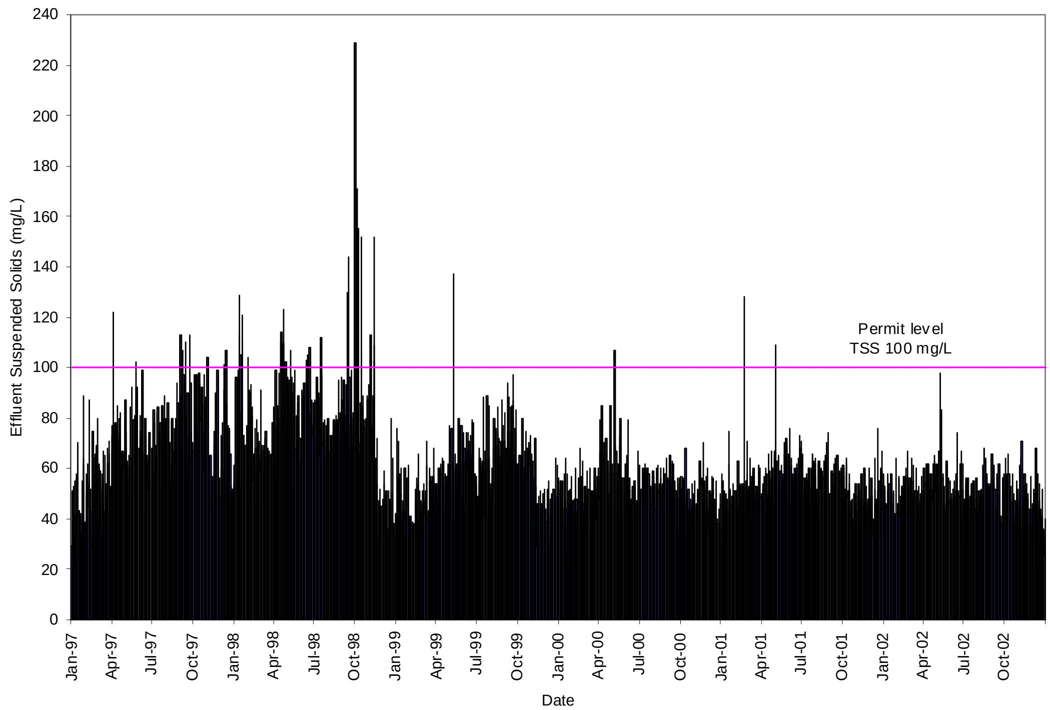


FIGURE 5.10
IIWWTP EFFLUENT TSS LOAD (1997 ~ 2002)

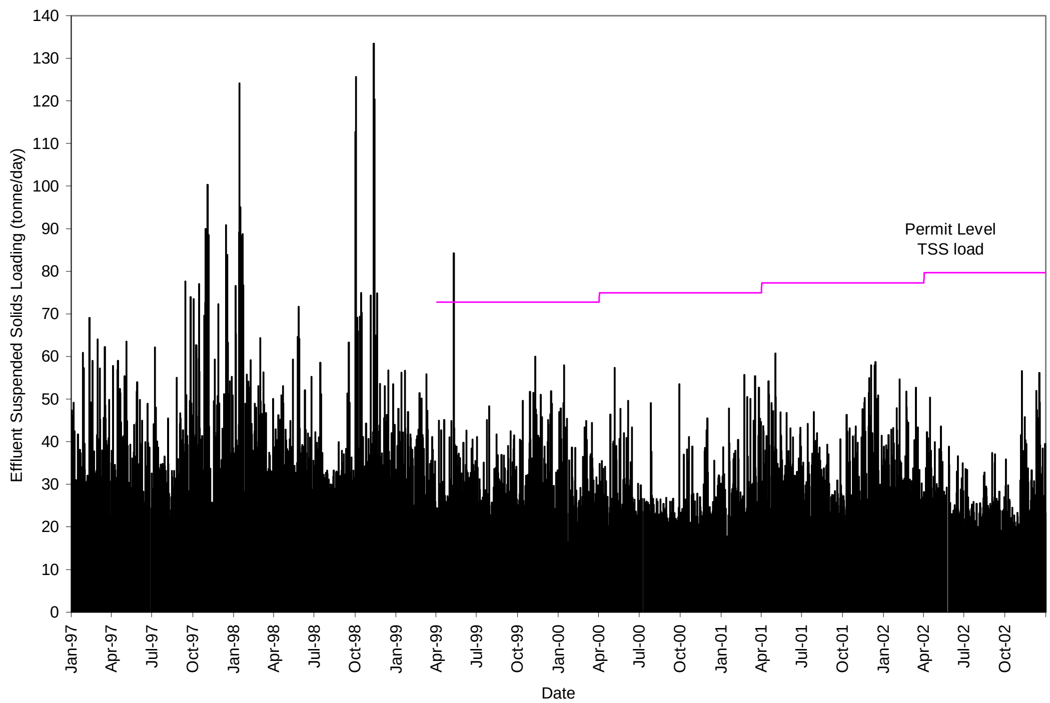


TABLE 5.7
IIWWTP EFFLUENT TOXICITY TEST RESULTS (1997 ~ 2002)

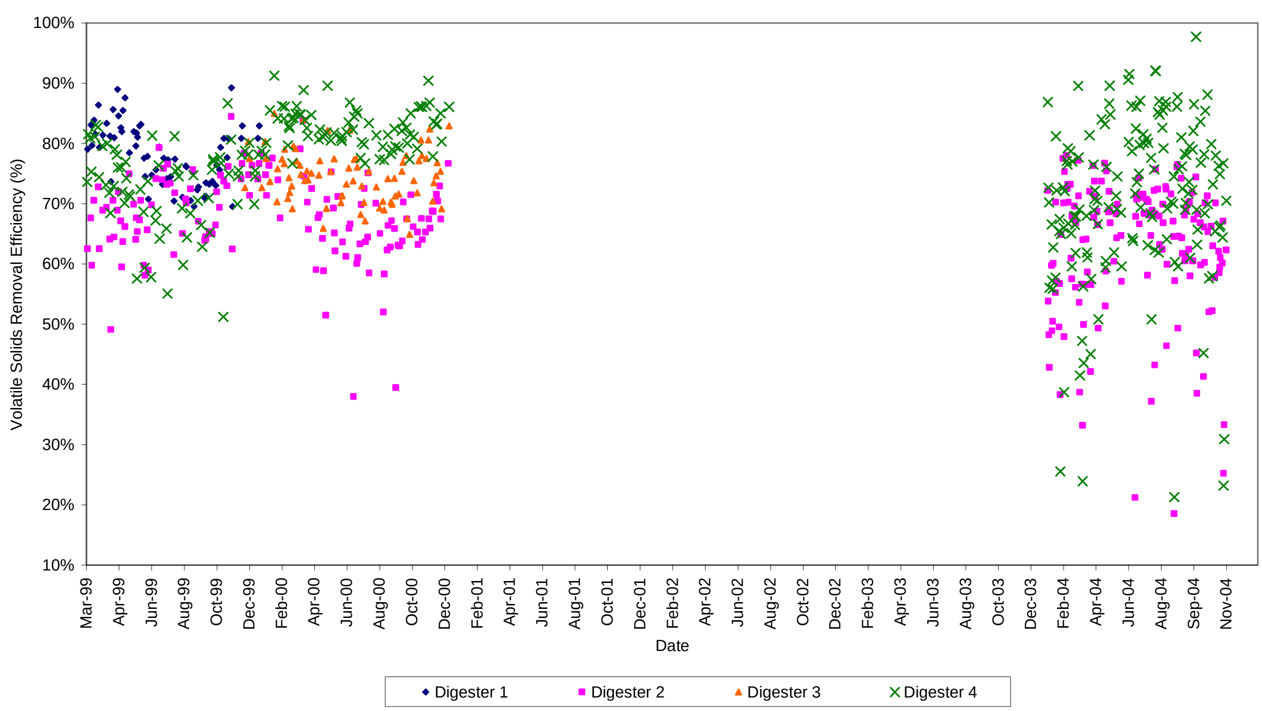
	96 hour LC_{50} (%V/V)												No. of Sample
Year/Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Failed Toxicity Test
1997	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	0 out of 12
1998	>100	>100	>100	94	>100	81	65	>100	95	>100	65,>100	>100	5 out of 13
1999	>100	>100	>100	>100	>100	63, >100	>100	82, 93	>100	>100	>100	>100	3 out of 14
2000	>100	>100	>100	82, 61	>100	>100							2 out of 7
2001	>100	>100	>100	>100	93	82	82	94	>100	>100	>100	>100	4 out of 12
2002	>100	>100	>100	>100	>100	98	>100	78	>100	>100	>100	>100	2 out of 12

5.1.4.2 <u>Solids Stream</u>

The solids stabilization efficiency is presented as volatile solids (VS) destruction efficiency in the digesters, from March 1999 to December 2000 and for 2004. Data for 2001 to 2003 is not available. The VS destruction efficiencies are shown in Figure 5.11, consistently achieving more than 50% of VS removal in 1999 and 2000, with single-stage operation. Plant staff has confirmed that the digester operation has been varied over the years and several system upsets have been experienced in some of the digesters. Single-stage or two-stage operation has been used alternately in the past years.

Currently (2004) the #1 and #3 are operated as the primary digesters, followed by # 2 and #4 as the secondary, respectively. Digester #1 was not in service most of the time during 2004 for maintenance and repair purposes. The VS destructions of 2004 shown in Figure 5.11 represent the overall removal efficiency of two-stage operation (i.e. VS differences between raw thickened primary sludge and the secondary digesters effluent). Single-stage operation (digester #2 only) achieved 10~20% less VS destruction than in the two-stage operation (digester #3 followed by #4). The overall VS destruction efficiencies were often found below 50%, which was primarily due to reduced digester capacity without digester #1. The pathogen kill efficiency in digester #2 was also reduced resulting in faecal coliform over 2,000,000 MPN/100mL. Further discussions of digester capacity issues are included in Appendix 7 of Interim Biosolids Handling Facilities.

FIGURE 5.11
IIWWTP DIGESTER VOLATILE SOLIDS DESTRUCTION EFFICIENCY
(1997 ~ 2000, and 2004)



5.1.5 Constraint on Upgrading and Expansion

At first glance, it appears that space is not a concern at IIWWTP, since the east and west portions of the property could be used for facility expansion. However, upon further analysis the following constraints must be taken into account. A site plan of the existing facility with property limits is included in Figure 5.12.

Sludge Stockpiles

The east portion of the site is low and has been used for dewatered sludge stockpile for over 30 years. The sludge stockpiles will have to be relocated prior to proceeding with site preparation for expansion for some of the interim upgrade options and for all options related to built-out to secondary. GVRD has indicated that sludge stockpiles will be relocated as required to accommodate plant expansion.

Fill and Pre-loading

The east portion of the site will require the placement of about 4.5 m of fill in order to raise the site and prevent flooding. In conjunction with placing fill, the site must be preloaded for a period of at least one year. It should be noted that the existing plant site has been preloaded prior to original construction over a 2-year period form 1959 to 1961. The plant expansion will also have to be preloaded prior to construction. However, in order to prevent settlement under existing structures, the pre-loading must be located at least 15 m from existing structures.

As a result of pre-loading setbacks, it appears that additional digesters will have to be located east or south (Further detail as to digester locations is provided in Technical Memoranda 12) of the existing plant instead of locating them west of the plant adjacent to the four existing digesters. Placing pre-loading west of plant could cause settlement of the berm around the sludge lagoon as well as under the effluent pump station, the maintenance building, the sludge thickener and the digesters. The requirements for pre-loading are discussed in more detail in the report by Trow Associates (Appendix 9).

Seismic Consideration

When subjected to an earthquake, the 15 meter thick layer of loose sand and gravel which underlain the site will behave like a heavy liquid. This will result in post-liquefaction consolidation settlement, loss of foundation bearing capacity and lateral spreading of the ground. To prevent lateral spreading, ground densification around the perimeter of the entire Iona Island treatment plant is proposed at an estimated cost of $1.7 million. Ground densification would consist of stone column (vibro-replacement) to 13 to 14 m depth on a triangular grid pattern at 2.8 m center-to-center spacing over a width of 15 m. The shores of the Fraser River have a high value for habitat (high productivity and diversity). The design of the ground densification will have to take into account environmental protection of the shoreline.

To prevent vertical movement at existing structures, soil anchors would be required. For any new addition, it is recommended that the footprint plus a 5 to 10 m wide area around the perimeter be densified.

Wetland

There are approximately 21 ha of land on the GVS&DD Iona Island WWTP property, which is located east of the existing plant. Approximately half of this property is covered by wetland. Any new structures that extend into the south half of the property may encroach on existing wetlands. New structures and tanks will be located on the north half of the site in order to minimize impact to the wetland. In order to minimize impacts on the wetlands, it is proposed to expand the plant on the north half of the property. However, if encroachment on the wetlands is necessary in order to accommodate the expansion, it may be necessary to provide some form of compensation for the loss of wetlands. This compensation could be a financial or by creating additional habitat in another location.

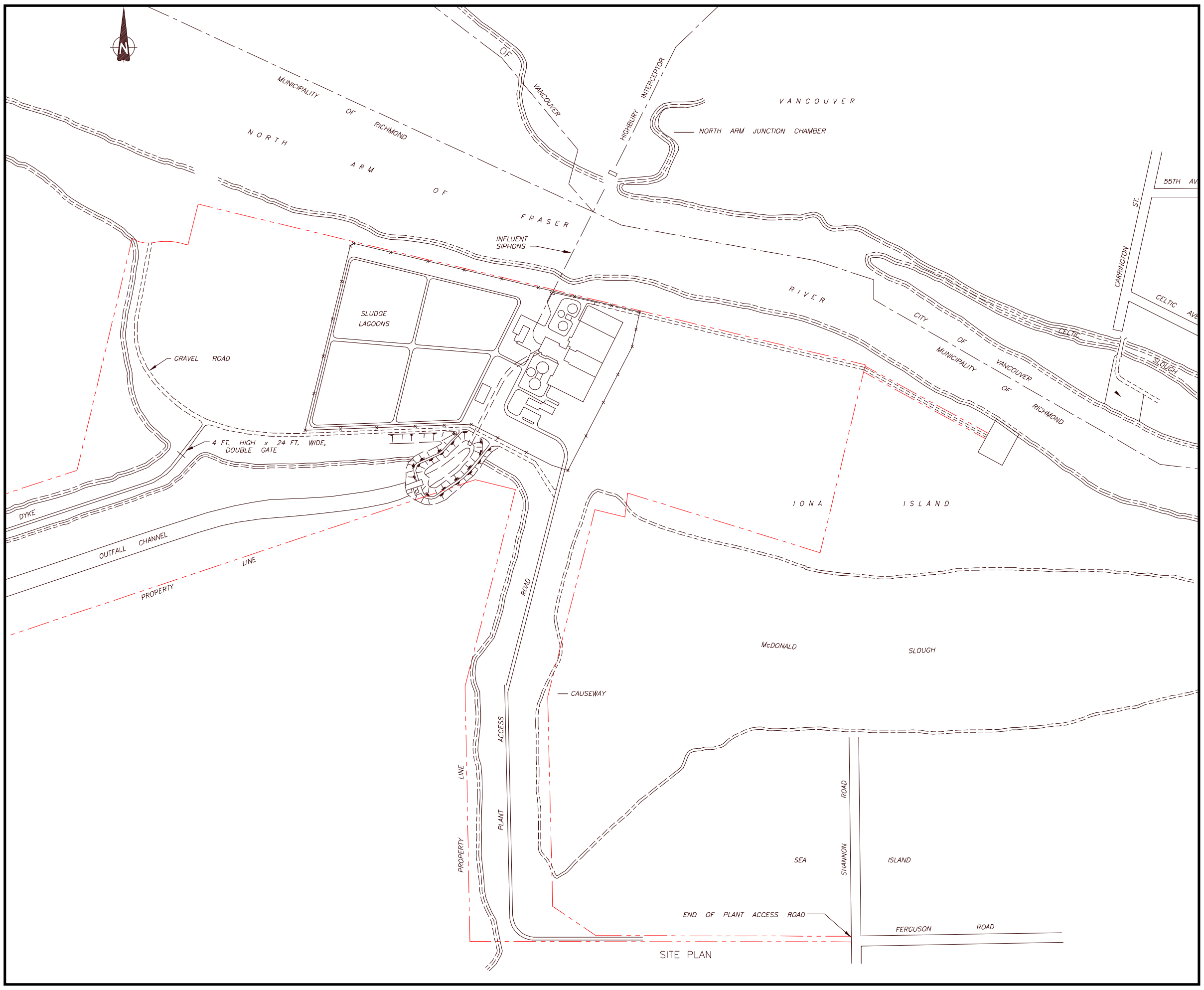
GVRD Parks

GVRD Parks has a maintenance yard located on the GVS&DD Iona Island WWTP property. The maintenance yard is located southeast of the primary sedimentation tanks. Depending on the footprint of the expanded plant, this maintenance yard may have to be relocated. GVRD Parks has advised that they would like to have a 50 to 75 m wide strip on the south side of the sewage treatment plant property along McDonald Slough. Also GVRD Parks has advised they would like to maintain the gravel road located north of the property as is allows access to the Parks property located east of the Iona Island WWTP property. It appears that the proposed upgrade will not impact the Parks property located east and south of the treatment plant site. Since no construction is proposed on most of the south half of the treatment plant property where the wetland is located, it appears that adding a 50 to 75 m strip of land to the Park property will be possible.

Shoreline of the Fraser River

There is an existing gravel road along the north side of the sewage treatment plant property along the Fraser River. The distance from the gravel road to the river is about 75 metres. The area south of the gravel road has been disturbed by sludge/biosolids storage while the area north the gravel road has been left in its natural state. It is proposed to locate any expansion of the treatment plant on land currently used for sludge storage and not to disturb the area between the gravel road and the shore of the Fraser River.

FIGURE 5.12
PROPERTY LIMITS OF IIWWTP SITE



5.2 LIONS GATE WWTP

5.2.1 General

The Lions Gate WWTP (LGWWTP) is a primary treatment plant that provides treatment of sanitary sewage from the whole north shore of Burrard Inlet from Horseshoe Bay to Deep Cove. This includes all of West Vancouver and North Vancouver. The first stage of the plant was built in 1960, and has under gone expansions in 1964,1975, and 1990.

The plant presently serves a population of approximately 174,000, and had an annual average daily flow of approximately 90 ML/d in 2002. Figure 5.13 shows the present site layout and property lines.

5.2.2 Process Description (See Figure 5.14)

The plant consists of a series of process units. Liquid train processes include coarse screening, influent pumping, pre-aeration, primary sedimentation, chlorine disinfection and dechlorination. Solids train processes include screenings dewatering, grit washing/dewatering, sludge thickening, digestion, and centrifuge dewatering.

5.2.2.1 Preliminary Treatment

The preliminary treatment processes consist of coarse screening, influent pumping, flow distribution and pre-aeration, grit removal.

Coarse Screening (See Figure 5.15)

Two mechanical bar screens with 6mm openings remove coarse solids from the influent flow. Screen capacity is designed for 171,000 m^3/d (2 m^3/sec) per unit. Captured screenings are dewatered and compacted and lifted to ground level in bins and hauled off site to landfill. Annual screenings in 2003 were 137 wet tonnes.

Influent Pumping (See Figure 5.16)

Four dry pit centrifugal pumps pump the screened sewage through four buried 510 mm and 610 mm discharge pipes into the influent distribution channel.

The combined pumping capacity is approximately 300,000 m^3/d. Raw sewage pumps No.1 & 2 are engine driven units using methane gas or natural gas and are rated at 1.03 m^3/sec at 14.3 m TDH. Pump No.3 is a DC motor with VFD speed drive rated at 0.96 m^3/sec, the smaller pump No.4 is AC motor driven with ASD drive rated at 0.45 m^3/sec.

FIGURE 5.13 LIONS GATE WASTEWATER TREATMENT PLANT - GENERAL SITE PLAN

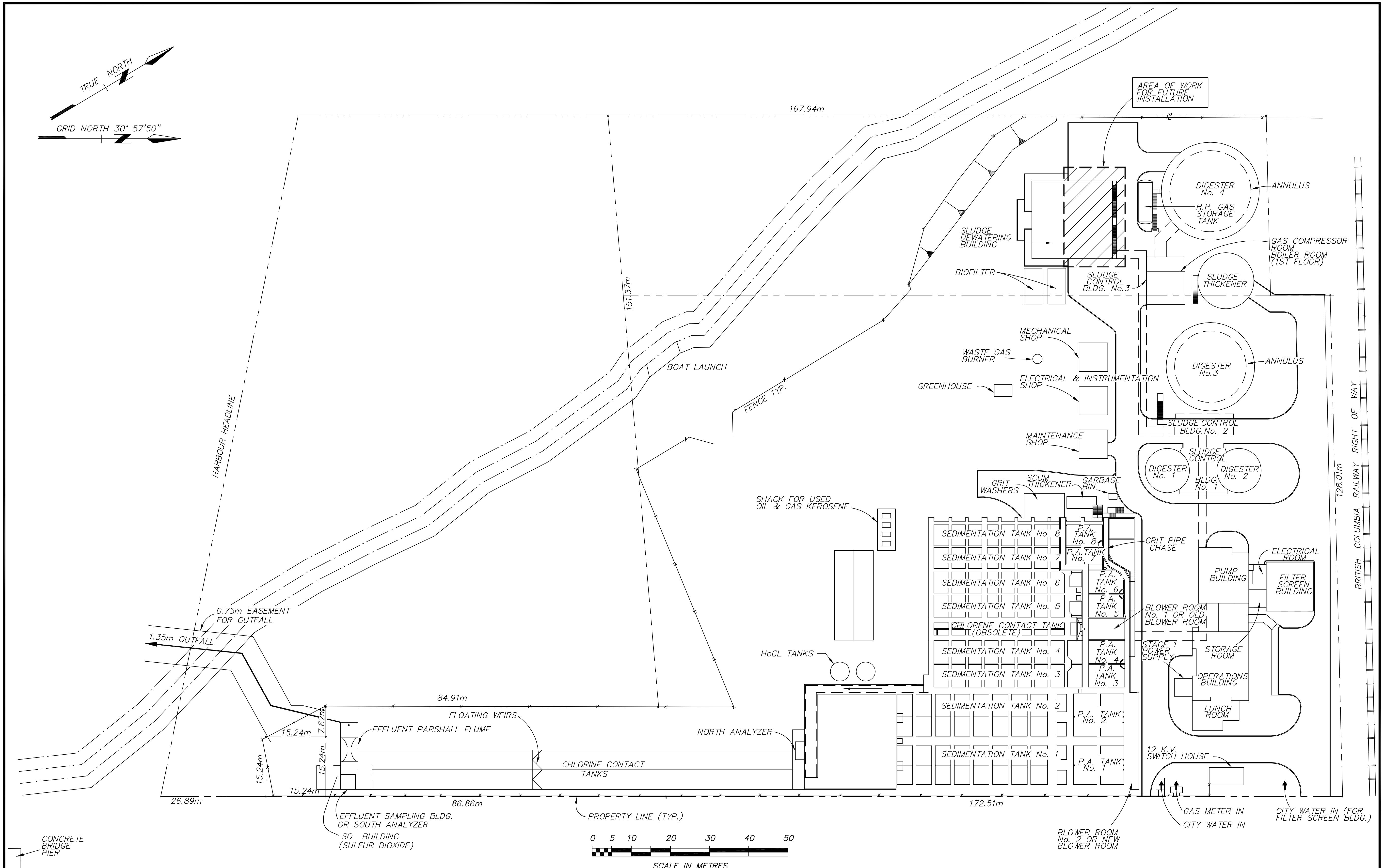


FIGURE 5.14 LIONS GATE WASTEWATER TREATMENT PLANT EXISTING PROCESS SCHEMATIC

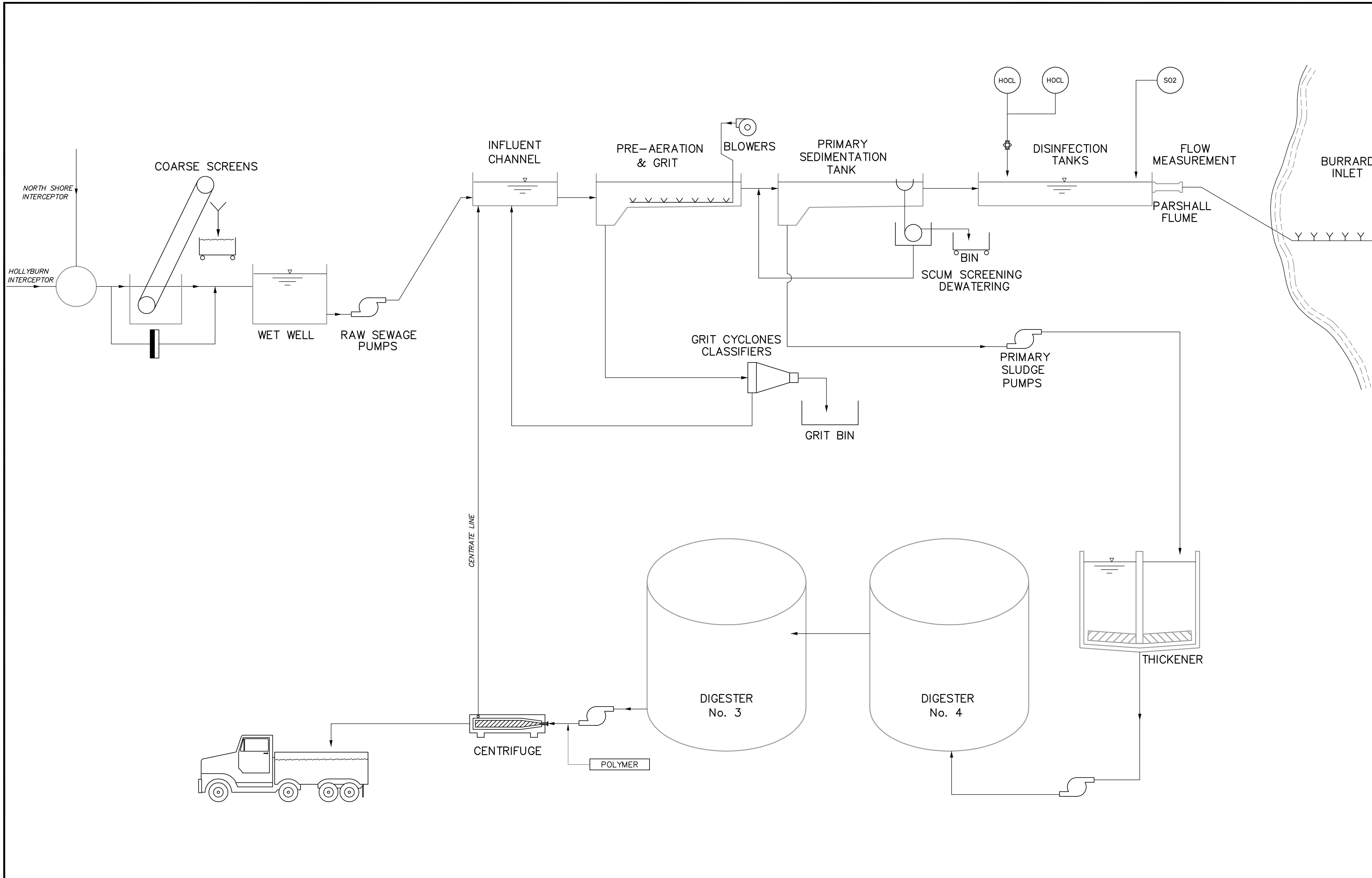


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FIGURE 5.15 LIONS GATE WASTEWATER TREATMENT PLANT HEADWORKS COARSE SCREENING SCHEMATIC

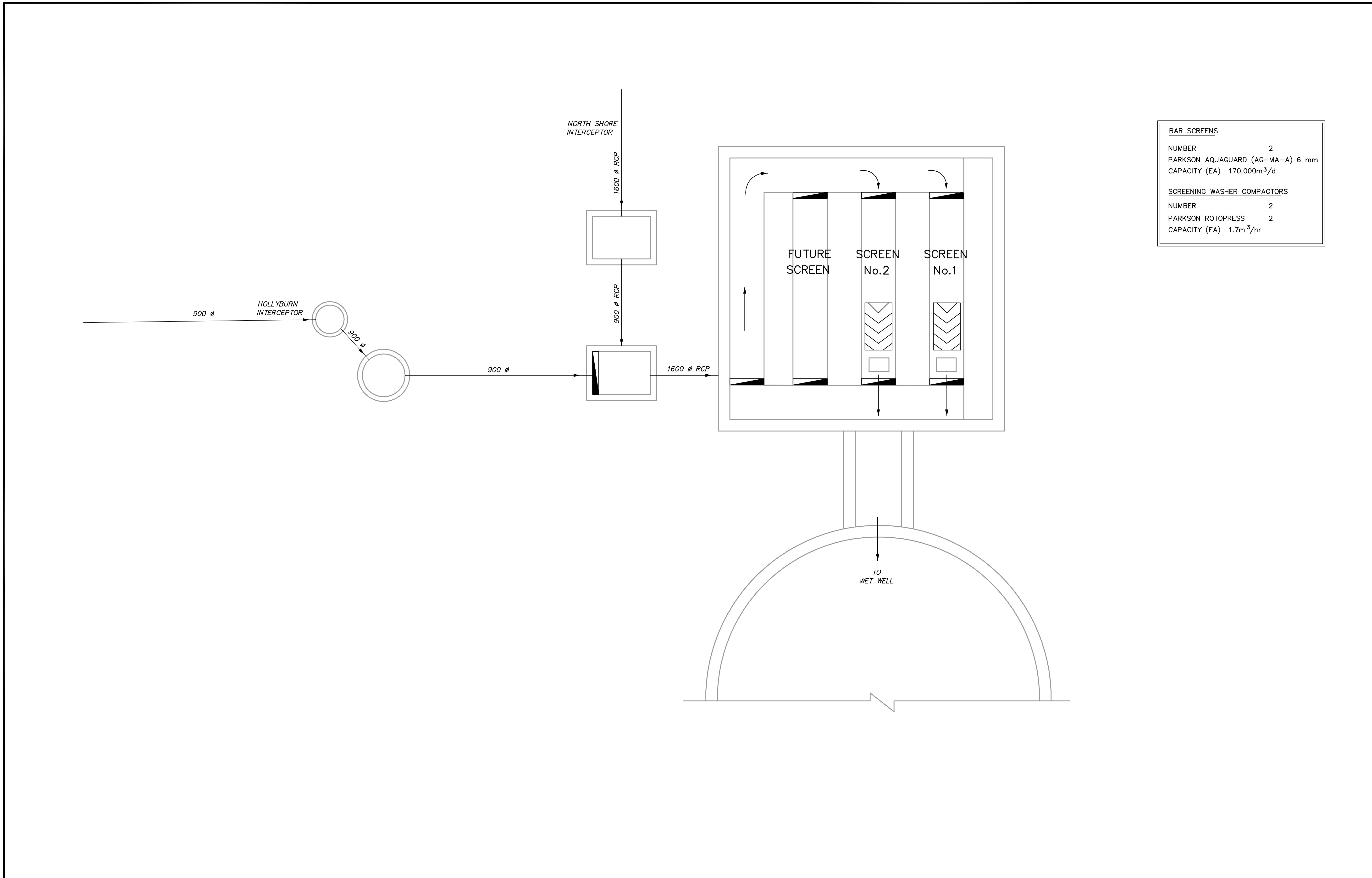

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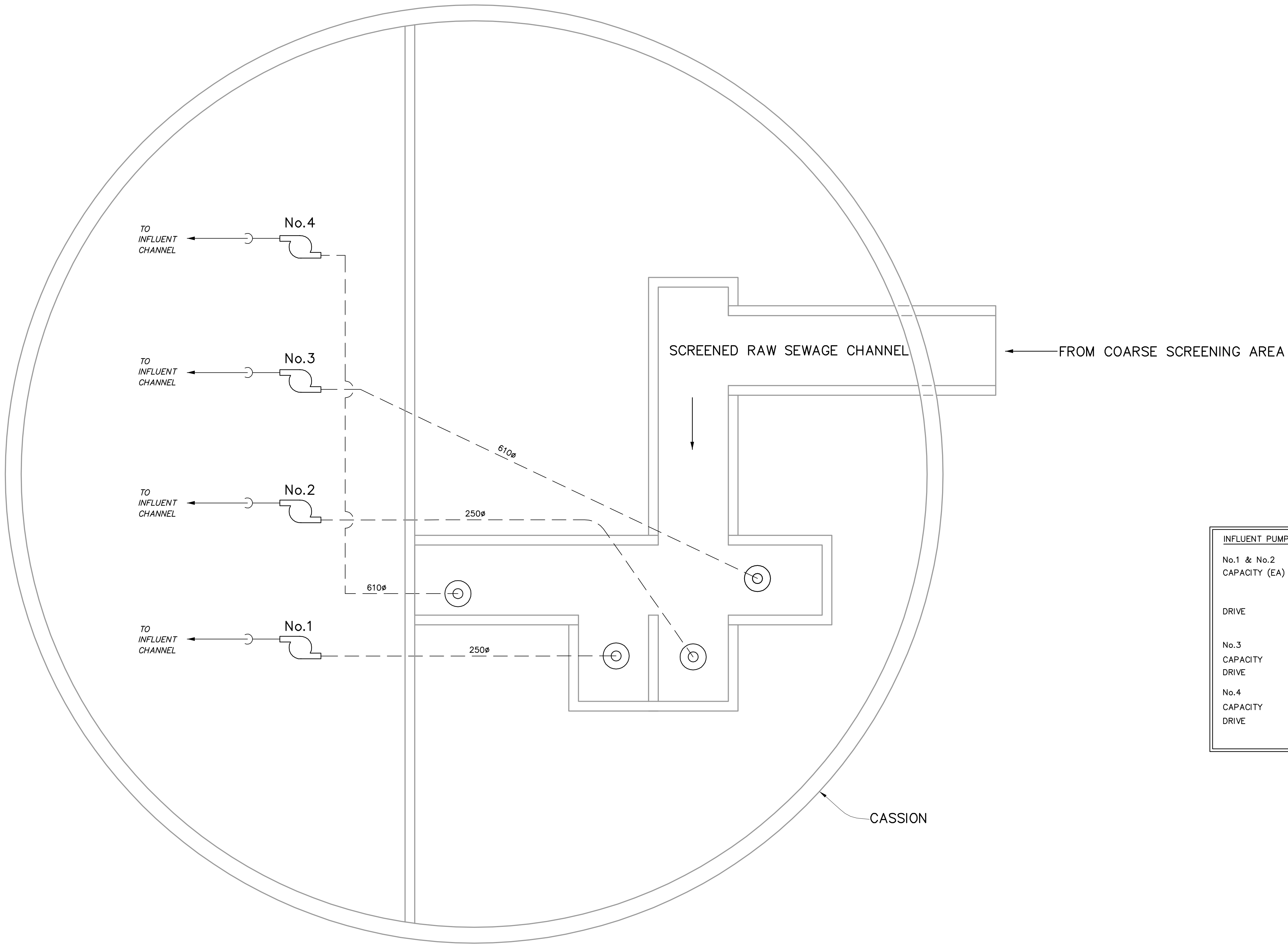
FIGURE 5.16 LIONS GATE WASTEWATER TREATMENT PLANT HEADWORKS INFLUENT PUMP SCHEMATIC



INFLUENT PUMPS	
No.1 & No.2	20"x20" FAIRBANKS MORSE
CAPACITY (EA)	90ML/d (16300 usgpm @ 43')
DRIVE	ENGINE DRIVE (NATURAL GAS OR METHANE)
No.3	BINGHAM
CAPACITY	–
DRIVE	DC 175kw MOTOR & VFD
No.4	FAIRBANK MORSE
CAPACITY	–
DRIVE	ELECTRIC MOTOR (W/ AC & VFD)

ACAD DWG. 415-1-103E 1:1 03-12-04

Flow Distribution Channel

This channel conveys the flow into the pre-aeration tanks through motorized sluice gates. Flap gates on the raw sewage pump discharges prevent backflow into the wet well.

Pre-Aeration & Grit Tanks (See Figure 5.17)

The pre-aeration grit tanks incorporate sloped floors and a roll aeration system to aerate the raw sewage and direct the settled grit to one side of the tank. Longitudinal screw conveyors direct the captured grit to hoppers where it is pumped out of the tanks by grit pumps to two grit cyclones. These grit cyclones separate and dewater the grit from the pumped flow. Grit is hauled off site and the cyclone water is returned to the headwork of the plant. The following Table 5.8 summarizes the dimensions of the 8 pre-aeration/ grit tanks.

TABLE 5.8
LGWWTP PREAERATION AND GRIT TANKS – DATA

Stage	**Tank No.**	**Dimensions**		**Area m^2**	**Depth (m)**	**Comments**
		L (m)	W (m)			
Stage Built	1A	14.3	5.79	83	4.06	Blower No. 5
3	1B	14.3	5.79	83	4.06	Blower No. 5
	2A	14.3	5.79	83	4.06	Blower No. 4
	2B	14.3	5.79	83	4.06	Blower No. 4
1	3	10.7	5.79	62	3.5	Blowers No. 1 and No. 2
	4	10.7	5.79	62	3.5	Blowers No. 1 and No. 2
2	5	10.7	5.79	62	3.5	
	6	10.7	5.79	62	3.5	
	7	10.7	5.79	62	3.5	Blower No. 3
	8	10.7	5.79	62	3.5	Blower No. 3
			TOTAL	704 m^2		

5.2.2.2 Primary Treatment (See Figure 5.18)

The primary treatment consists of the primary sedimentation tanks.

Primary Sedimentation Tanks

Flows from the pre-aeration/grit tanks are introduced into the primary sedimentation tanks through a series of submerged openings. The low velocities in the sedimentation tanks allow suspended solids to settle out under gravity. Longitudinal tank sludge scrapers direct the settled solids to primary sedimentation tank sludge collection hoppers located at the front of the tanks. Raw sludge is then pumped out by the raw sludge pumps to the sludge thickener tank.

FIGURE 5.17 LIONS GATE WASTEWATER TREATMENT PLANT EXISTING PREAERATION TANKS SCHEMATIC

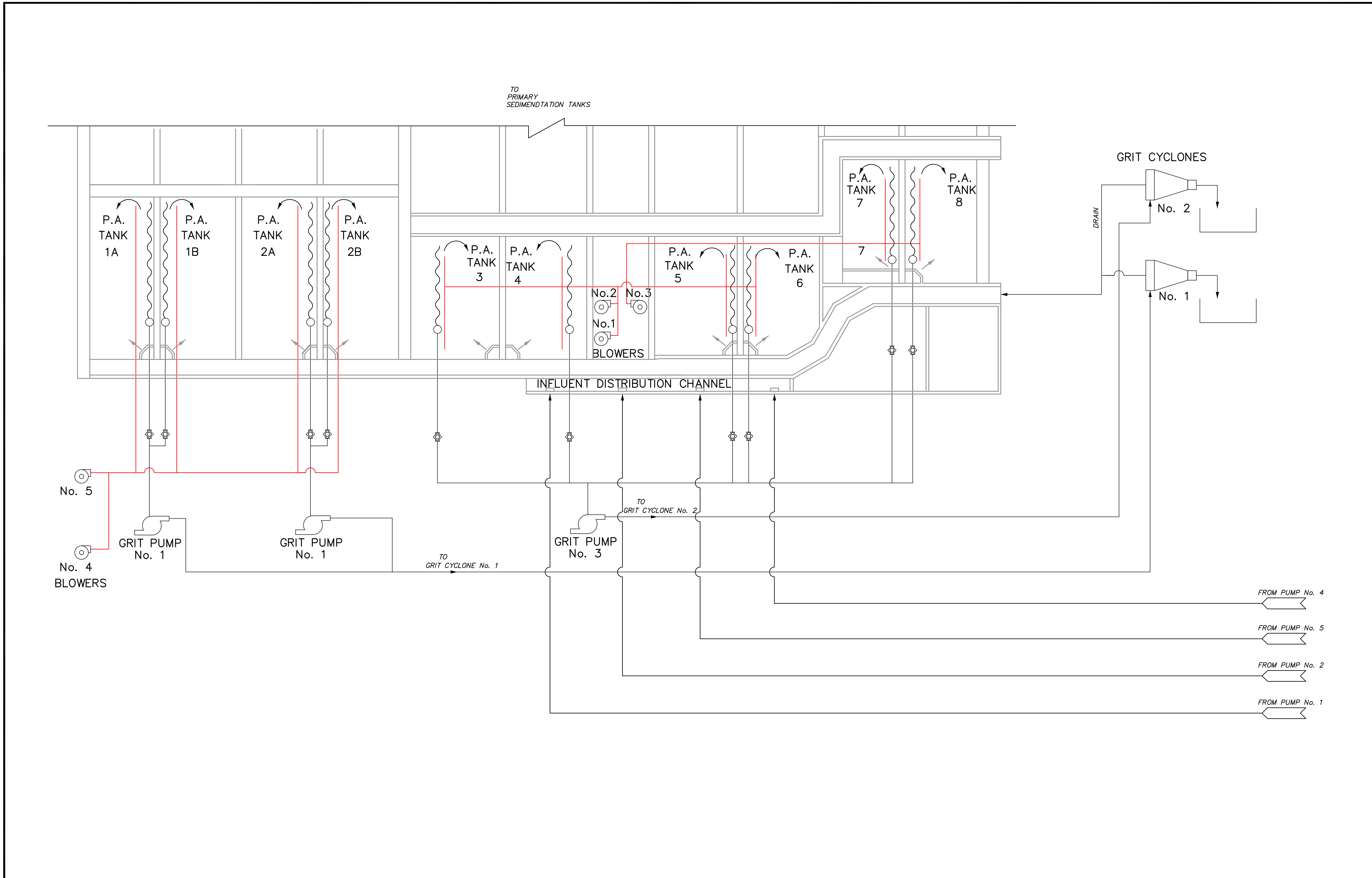


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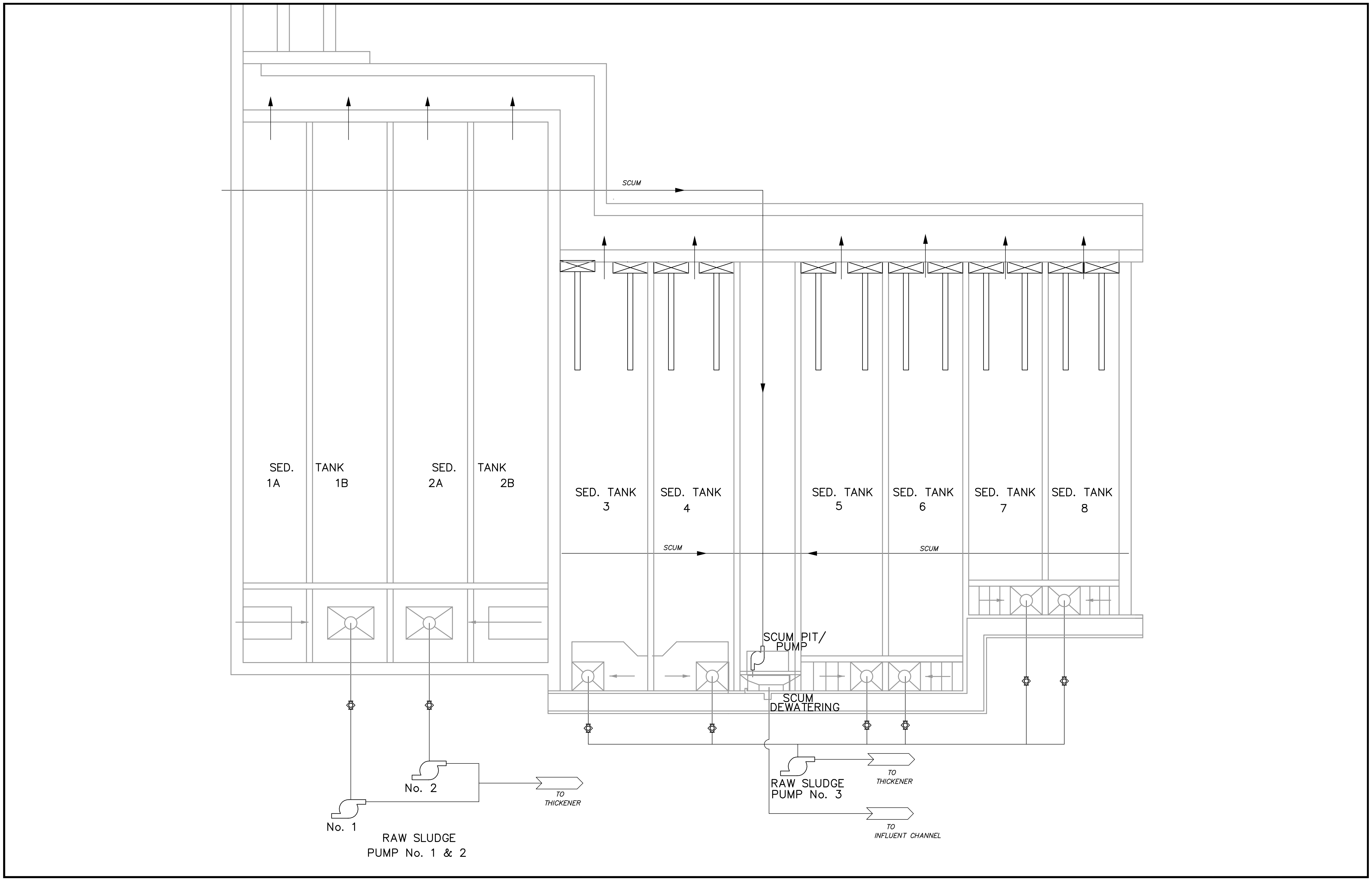
FIGURE 5.18 LIONS GATE WASTEWATER TREATMENT PLANT EXISTING PRIMARY SEDIMENTATION SCHEMATIC



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Scum troughs at the back of primary sedimentation tanks 1 & 2 collect scum directed to the trough by the surface level return flights of the longitudinal sludge scrapers. Hydraulic actuators tip the scum troughs periodically to skim off the scum. In primary tanks 3 through 8, water nozzles direct the scum to the front of the tanks where surface skimmers skim of the scum.

All scum is collected in a scum pit and is pumped out into a two-stage drum dewatering system. Dewatered scum is collected in a bin and hauled offsite. Scum dewatering underflow is directed back into the influent channel.

The primary settled wastewater is collected by surface launderers flowing into rectangular channels, which flow into the main concrete channel leading to the chlorine contact tanks. Table 5.9 summarizes the size of the eight (8) primary sedimentation tanks.

TABLE 5.9
LGWWTP PRIMARY SEDIMENTATION TANKS

Stage	Tank No.	L m	W m	Area m^2	Depth m	Volume m^3
3	1A	64.6	5.79	374.2	2.9	1085
	1B	64.6	5.79	374.2	2.9	1085
	2A	64.6	5.79	374.2	2.9	1085
	2B	64.6	5.79	374.2	2.9	1085
1	3	37.8	5.79	218.8	2.74	600
	4	37.8	5.79	218.8	2.74	600
2	5	37.8	5.79	218.8	2.74	600
	6	37.8	5.79	218.8	2.74	600
	7	32	5.79	185.3	2.74	508
	8	32	5.79	185.3	2.74	508
TOTAL				2743 m^2		7756 m^3

Disinfection and Dechlorination (See Figure 5.19)

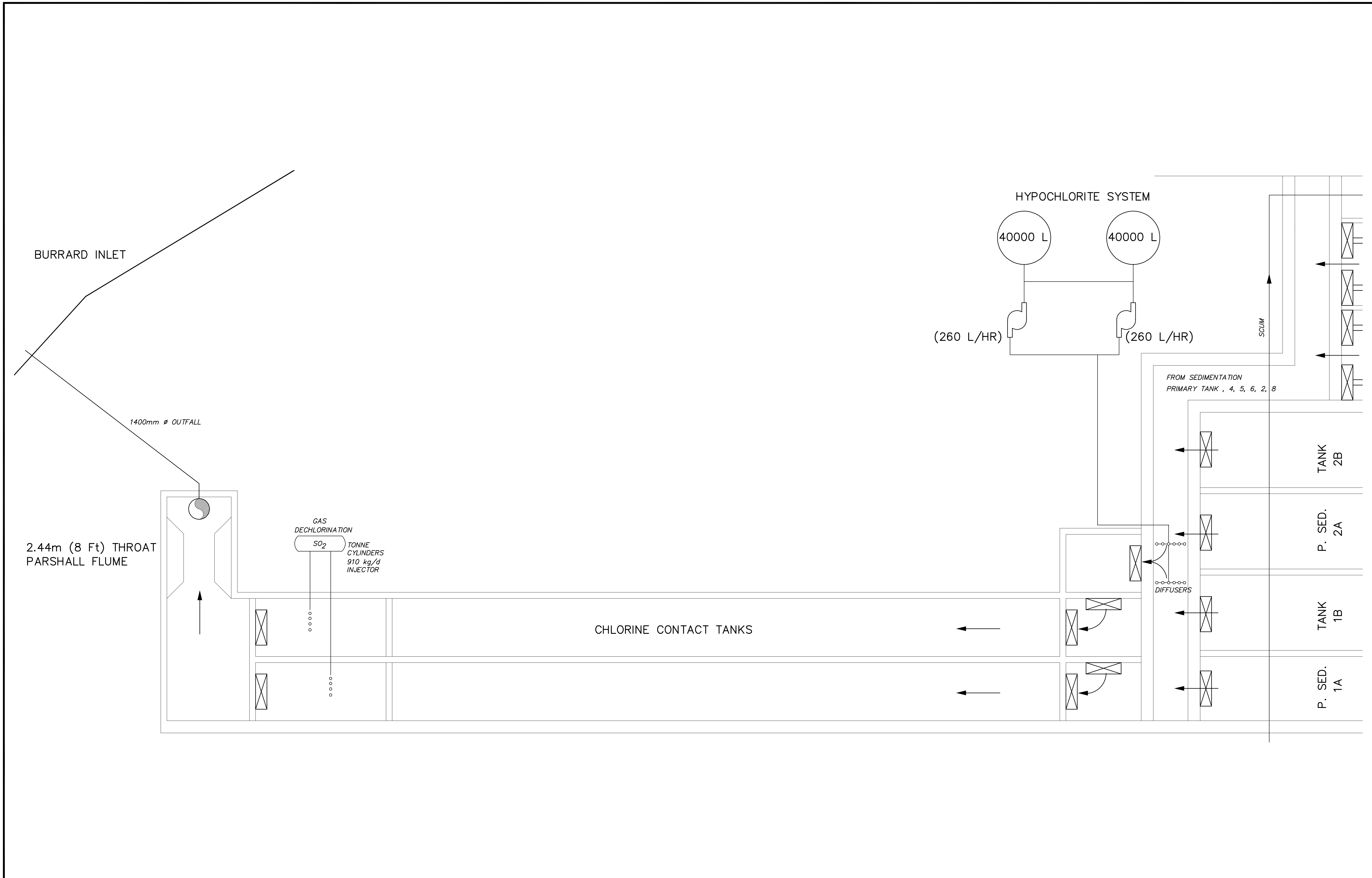
In the summer months, hypochlorite solution is dosed into the front end of the two chlorine contact tanks. After traveling through the tank the plant effluent is de-chlorinated with sulphur dioxide gas.

Finally the plant effluent passes through an 2.44 m throat Parshall flume and is discharged into Burrard Inlet via a 1370 mm diameter pipe. Table 5.10 summarizes the main characteristics of these tanks, flume and outfall.

TABLE 5.10
LGWWTP CHLORINE CONTACT TANK AND OUTFALL

Stage	Tank No.	L m	W m	Area m^2	Depth m	Volume m^3
3	No. 1	108.2	4.57	495	3.84	1900
	No. 2	108.2	4.57	495	3.84	1900
	Effluent Parshall Flume size 2.44 m (8ft) throat free flow capacity = 0.13 ~ 3.95 m^3/sec					
2	Outfall size1370 mm (54inch) dia. concrete flow rate @ extreme HHWL = 2.52 m^3/sec					

FIGURE 5.19 LIONS GATE WASTEWATER TREATMENT PLANT



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5.2.2.3 Solids Handling (See Figure 5.20 ~ 5.22)

Solids train processes include screenings dewatering, grit washing and dewatering, sludge thickening, digestion, centrifuge.

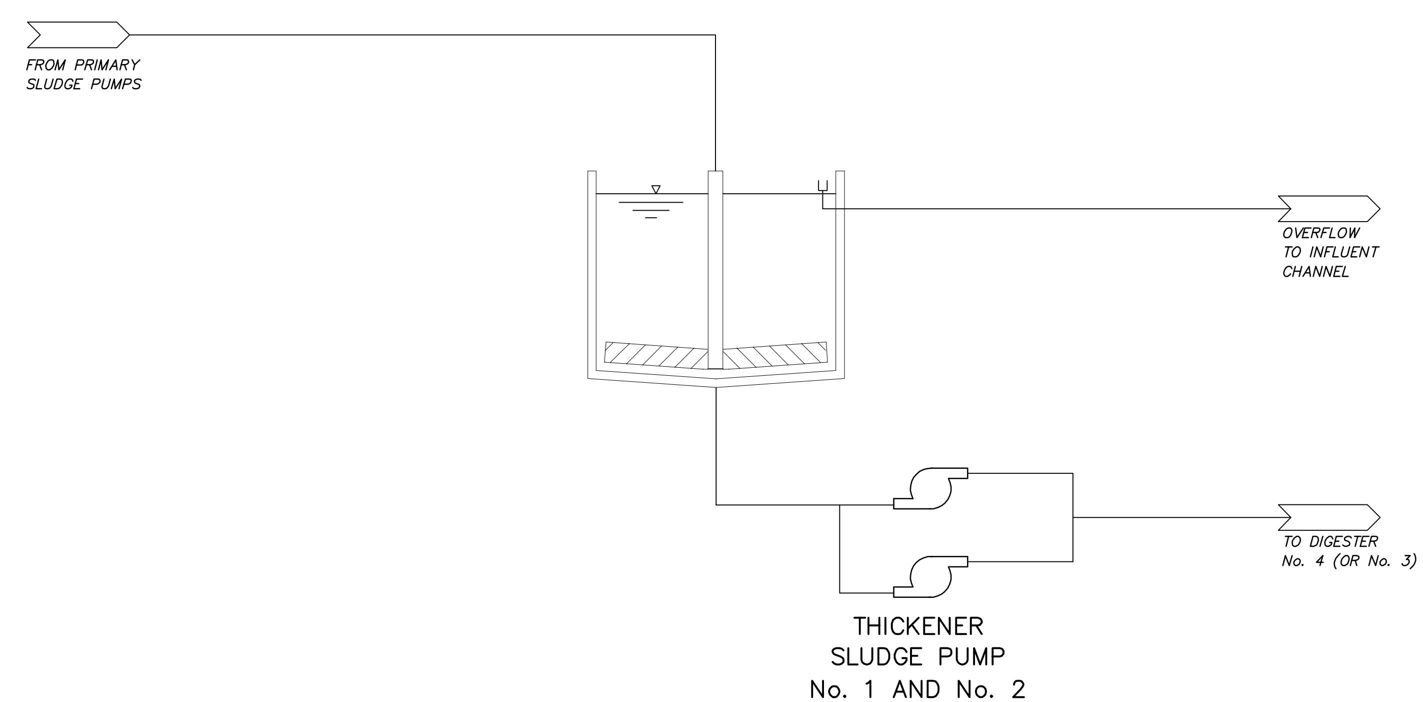
Sludge Thickener (Figure 5.20)

One gravity thickener receives sludge from the primary sedimentation tanks. The thickener, circular in shape, consists of an influent well and a rotating sludge collector. Solids settle to the bottom by gravity and are thickened by the settling and compaction of the solids. The thickened sludge is pumped to the digesters via heat exchange loops. Thickener overflow and scum is pumped back to the influent distribution channel and returned to the main plant flow. The dimensions and design criteria of the gravity thickener is summarized in Table 5.11.

TABLE 5.11
LGWWTP SLUDGE GRAVITY THICKENER

Stage 3	Number:	1
	Diameter:	13.7 m
	Side Water Depth:	3.05 m
	Overflow Rate:	~ 3.0 m^3/m^2d
	Raw Sludge Concentration:	0.5 – 2%
	Thickened Sludge Concentration:	4-5%
	Raw Sludge Pump Number:	3
	Thickened Pump Number:	2

FIGURE 5.20 LIONS GATE WASTEWATER TREATMENT PLANT
THICKENER SYSTEM



ACAD DWG. 415-1-1030 1:1 03-12-15

<u>Anaerobic Digesters (Figure 5.21)</u>

The plant has two operating anaerobic digesters, with digesters #4 and #3 operating in series. Old digesters #1 and #2 are no longer used. Thickened sludge from the sludge thickener and thickened scum from the scum thickener are stabilized in the digesters and undergo thermophilic digestion at approximately 55ºC. Digester sludge is recirculated throughout the tank by recirculation pumps and gas mixing systems. Off-gas produced is used by the gas engine driven, raw sewage pumps No. 1 and No. 2, burnt for digester heating and the excess is flared. Digester gas storage tank provides 283 m^3 (10000 ft^3) of storage and is rated for 125 psi. The design gas production is estimated at approximately 0.64 m^3/kg VSS reduced, producing approximately 4900 m^3 of gas per day. The dimensions and design criteria of the digesters are summarized in Table 5.12.

TABLE 5.12
LGWWTP DIGESTER SYSTEM

Stage			
2 (1964)	Digester No. 3	Number	1
		Diameter	19.8 m
		Side Water Depth	10.0 m
		Solids Loading	1.92 kg VSS/m^3/30-day month
		Hydraulic Retention Time (operating in series)	12 days @ 6% sludge solids
		Average Temperature	55ºC
		Recirculation Pumps	2 ~ 6.3 L/s @ 15.2 m TDH
3 (1975)	Digester No. 4	Number	1
		Diameter	19.8 m
		Side Water Depth	10.0 m
		Solids Loading	1.92 kg VSS/m^3/30-day month
		Hydraulic Retention Time (operating in series)	12 days @ 6% sludge solids
		Average Temperature	55ºC
		Recirculation Pumps	2 – 6.3 L/s @ 15.2 m TDH

<u>Centrifuge Dewatering (Figure 5.22)</u>

Two Alfa-Laval Sharples Centrifuges dewater digested sludge from 3-4% TS to 30-35% TS. Digested sludge is batch pumped from digester No. 3 to either one of the centrifuges. The sludge passes through an inclined macerator, dosed with polymer then fed to the centrifuge. Dewatered biosolids are conveyed into trucks and hauled offsite for disposal. Centrate flows by gravity back into the influent distribution channel. Table 5.13 outlines the key centrifuge characteristics.

FIGURE 5.21 LIONS GATE WASTEWATER TREATMENT PLANT DIGESTERS

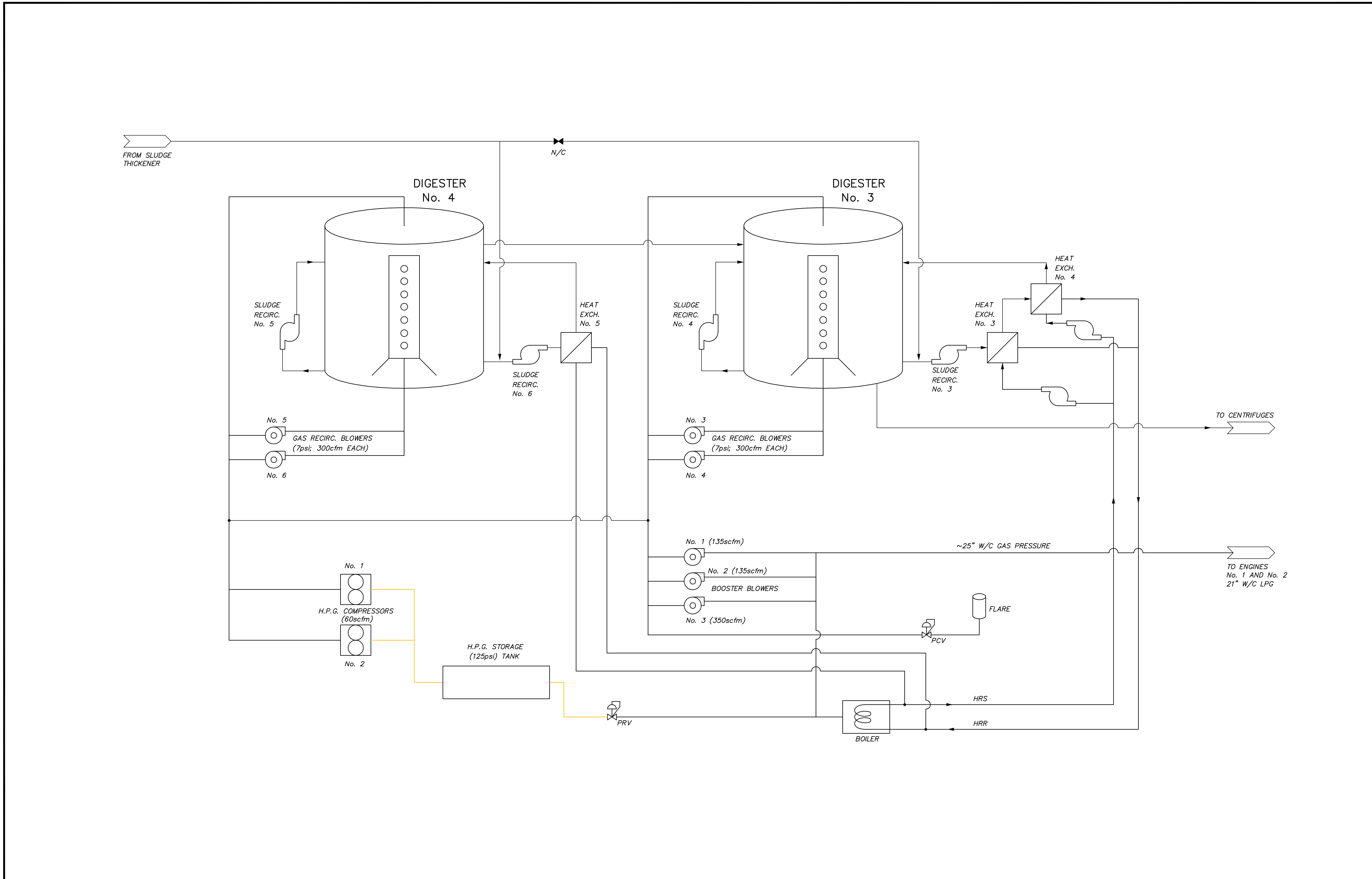


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FIGURE 5.22 LIONS GATE WASTEWATER TREATMENT PLANT DEWATERING SCHEMATIC

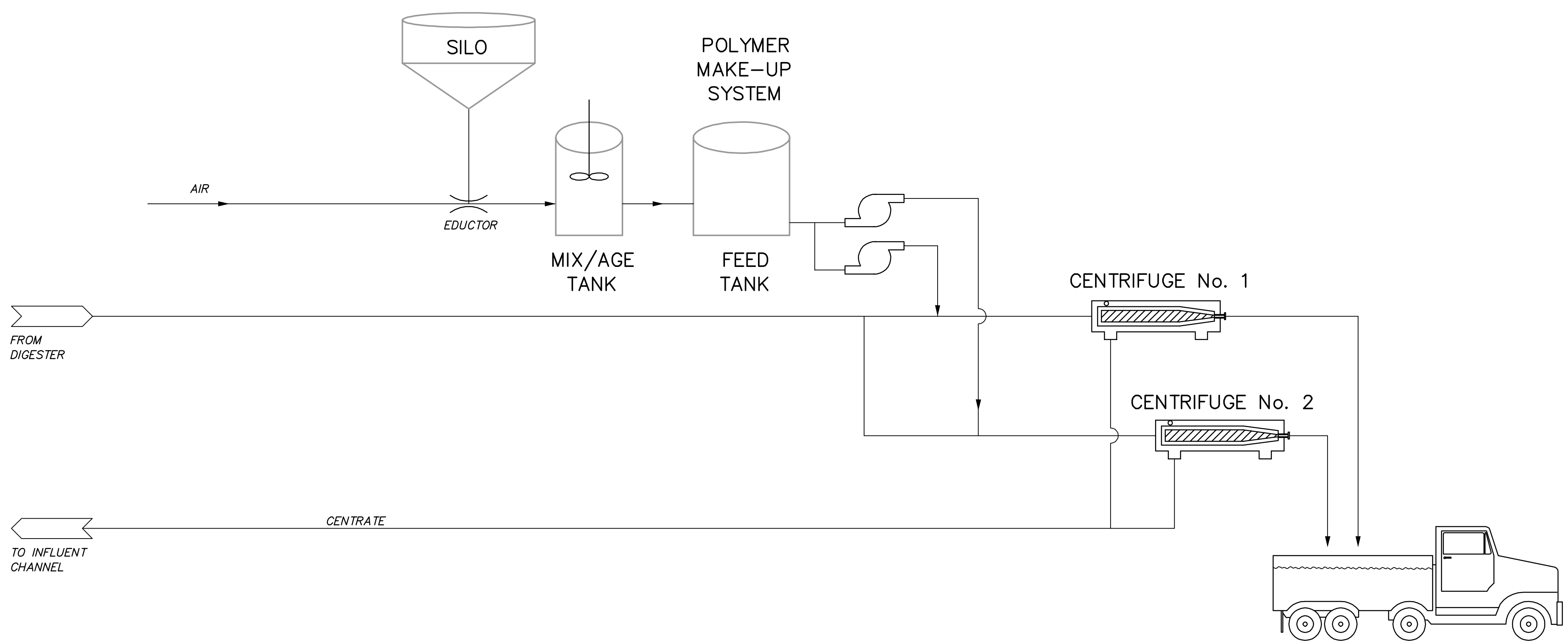


TABLE 5.13
LGWWTP DEWATERING SYSTEMS

Stage 4	Polymer System:
	• Aged Polymer Tank, 83 NL, 0.5% polymer, 45 minutes aging
	Centrifuge No. 1 and No. 2:
	• Capacity – 80-100 L/min
	• Feed TSS – 2-3%
	• Cake TSS – 30-35%

5.2.3 Current Facility Capacity

The Lions Gate WWTP design flows and loads are summarized in Table 5.14. For comparison, the 2002 average plant loadings and performance are also listed. I&I is a major problem the collection system as evidenced by the largely domestic type, weak strength sewage, and very high PWWF value, which is almost 3 times the ADWF.

TABLE 5.14
LGWWTP DESIGN FLOWS/LOADS AND 2002 AVERAGES

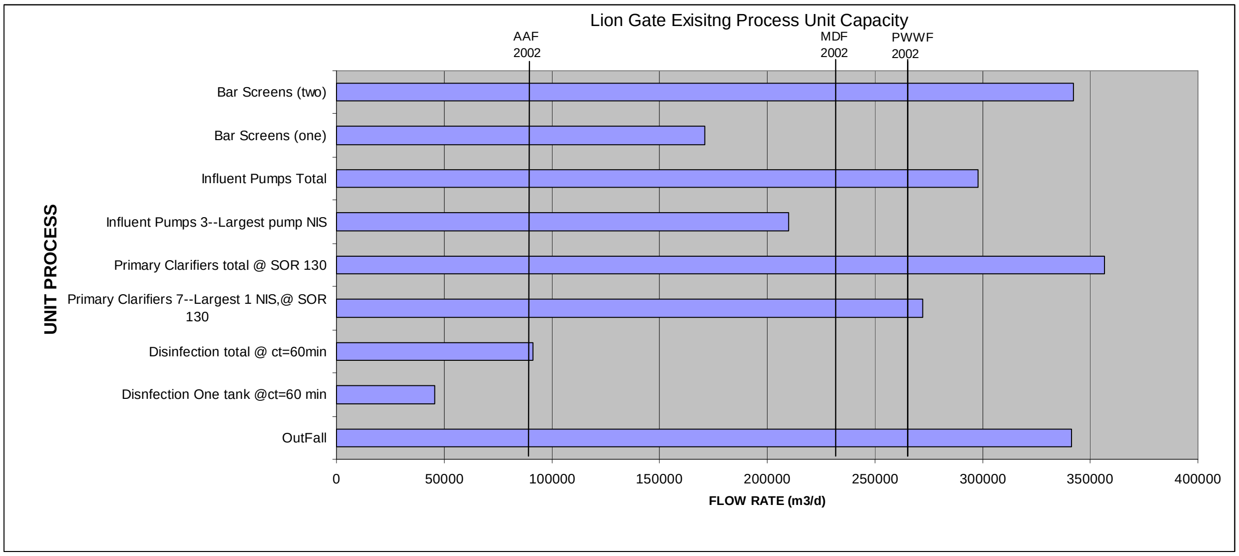
Parameter	Unit	Design Value		2002 Average	
		Total	Per Capita[1]	Total	per Capita[2]
Population	person	204,000	-	175,000	-
ADWF	m^3/d	102,000	500 L/c/d	-	-
AAF	m^3/d	-	-	89,760	513 L/c/d
MMF	m^3/d	-	-	106,000	-
MDF	m^3/d	-	-	231,400	-
PWWF	m^3/d	217,750	-	264,000	-
Influent BOD	mg/L	250	-	140	-
Influent BOD	kg/d	25,400	0.125 kg/c/d	12,600	0.072 kg/c/d
BOD removal	%	35%		35%	-
Influent TSS	mg/L	250		168	-
Influent TSS	kg/d	25,400	0.125 kg/c/d	15,100	0.086 kg/c/d
TSS removal	%	60%	-	68%	-
Effluent BOD	mg/L	-	-	91	-
Effluent BOD	kg/d	-	-	8,200	-
Effluent TSS	mg/L	-	-	53	-
Effluent TSS	kg/d	-	-	4,800	-

1 From Stage 3 Design Data (Drawing G-42)
2 From plant 2002 Data

5.2.3.1 <u>Liquid Train</u>

Figure 5.23 shows the hydraulic design capacities of the major liquid train processes with all units in operation. It also shows the design hydraulic capacity with the largest capacity unit in a given unit process not in service. Noted deficiencies in existing plant flow conditions are influent pumping capacity, and screening. We note that disinfection is only required from April to October, and that the 60 minute contact time capacity is shown for information purposes, should this be required. The primary sedimentation tank capacity is shown at the assumed SOR at PWWF of 130 $m^3/m^2/d$.

FIGURE 5.23
LGWWTP EXISTING PROCESS UNIT CAPACITIES



It is assumed that the flow distribution to each primary sedimentation tank is proportional to the tank surface area. The past known problem of an imbalance seems largely to have been resolved with the installation of the control gates on the entrance to the pre-aeration tanks, and level monitoring on the discharge weirs of each primary sedimentation tank. Table 5.15 lists the key characteristics of the primary sedimentation tanks.

TABLE 5.15
SURFACE OVERFLOW RATE AND WEIR OVERFLOW RATE OF LGWWTP PRIMARY SEDIMENTATION

Parameters	Unit	Design	2002 Data	Typical Design Range
SOR at ADWF	$m^3/m^2/d$	37	36	32-48
SOR at PWWF	$m^3/m^2/d$	130	96	80-120
WOR at ADWF	$m^3/m/d$	-	-	-
WOR at PWWF	$m^3/m/d$	-	~564	124-496

5.2.3.2 Disinfection and Outfall

The disinfection system was upgraded from a chlorine gas system to a hypochlorite system in early 2003. Capacity of the new dosing pump is 260 L/hr, which equates to approximately 28 l/hr of hypochlorite at 11% solution strength. Two (2) 40,000 litre tanks store the hypochlorite solution.

The outfall (54 inch) 1.37 m pipe was built in 1975 as part of the Stage 3 expansion. Design capacity at the extreme high water level in Burrard Inlet is 207,000 m^3/d (89 cfs). The 2002 PWWF event of 264,000 m^3/d indicates that upgrading of this outfall should be investigated.

5.2.3.3 Solids Train

The design capacities of the thickener and digesters are identified in Table 5.16.

TABLE 5.16
LGWWTP THICKENER AND DIGESTER SOLIDS AND HYDRAULIC LOADS

Parameters	Units	Design	2002 Averages	Typical Design Range
Thickener Solids Load (at 3.5% TS)	$kg/m^2/d$	103	~68.5	87-136
Thickener Overflow Rate (set by feed pumps)	$m^3/m^2/d$	29.5	~4.4	--
Digester Solids Load	$kgVSS/m^3/d$	1.92	1.63	1.6-4.8
Digester Detention Time (both digester 3 and 4)	day	24	18.0	10-20
Digester Feed Conc.	%	6	6.1	5-7
Digester Feed Flow Rate (only Digester No. 4 in use @ 2002)	m^3/d	255.3	200	--

The thickener feed pumps are operated sequentially based on a timer. Each feed pump is rated at about 325 gpm. Therefore the thickener operating SOR is 14.4 $m^3/m^2/d$.

The thickener has however settled on one side, and the SOR is not equally distributed as only about ¼ of the perimeter tank overflow weir discharges flow. Despite this, and due to the low feed rate, SOR is not a present process limitation. Lack of thickener redundancy is a process concern.

The two high rate, thermophilic digesters operate in series and at about 55ºC. They were originally designed for conventional mesophilic digestion, but have been operated at thermophilic temperatures since February 1991. Typical SRT for high rate stabilization is 15 days. At present, the digestion system can handle the plant solids loading, but without redundancy. With 1 digester out of service, (as per the past 2 years due to the refurbishing of Digester No. 4 and currently Digester No. 3) the 2002 solids loading is about 3.26 kg/m^3/d, which is on the high side of the design range.

5.2.3.4 Dewatering System

The current design capacity of the centrifuge dewatering system is identified in Table 5.17.

TABLE 5.17
LGWWTP CENTRIFUGE DEWATERING CAPACITY

	Units	Design
Centrifuge Feed Rate (each)	m^3/hr	480-600
Centrifuge Solids Feed Rate, % TSS (each)	%	2-3

One of the two centrifuges operates approximately 30 hours per week on average.

5.2.3.5 Other Plant Issues

One concern with the entire plant (liquid and solids trains) is the limited effectiveness of the grit removal system. Large amounts of grit are settled out in the primary sedimentation tanks and pass into the digesters and dewatering systems. This grit ends up settling in the digesters, and wears out the centrifuge scroll and bowl. Further, heavy influent grit loadings during high flow events trip out the mechanical coarse screens.

A greater concern is the lack of redundancy in the solids handling system. Digester 1 is currently being used as a digested sludge storage tank. This vessel is very old, does not have adequate mixing, and does not have any gas/odour control. If it is not in service there is no effective way to feed sludge to the centrifuges as the digesters do not have fill and draw capacity. Without a digested sludge storage tank, Lions Gate would have to truck digested sludge to other plants.

5.2.4 Effluent Quality

Liquid Stream

In accordance with the permit requirements, the effluent quality is presented as Daily Total Effluent Flow, daily BOD and TSS concentrations, daily BOD and TSS loads, from January 1997 to December 2002. The daily total effluent flows are shown in Figure 5.24, with a maximum flow of 256,700 m^3/day recorded in March 1997. The effluent BOD concentrations and loads are shown in Figure 5.25. In 1998 the permitted effluent BOD concentration was exceeded 4 times. The effluent TSS concentrations and loads are shown in Figure 5.26. No violation of the TSS concentration or load was reported in 2002. The effluent toxicity LC_{50} test results are summarized in Table 5.18. Plant effluents have failed the toxicity test on occasion over the last 5 years.

FIGURE 5.24
LGWWTP DAILY TOTAL EFFLUENT FLOW (1997 ~ 2002)

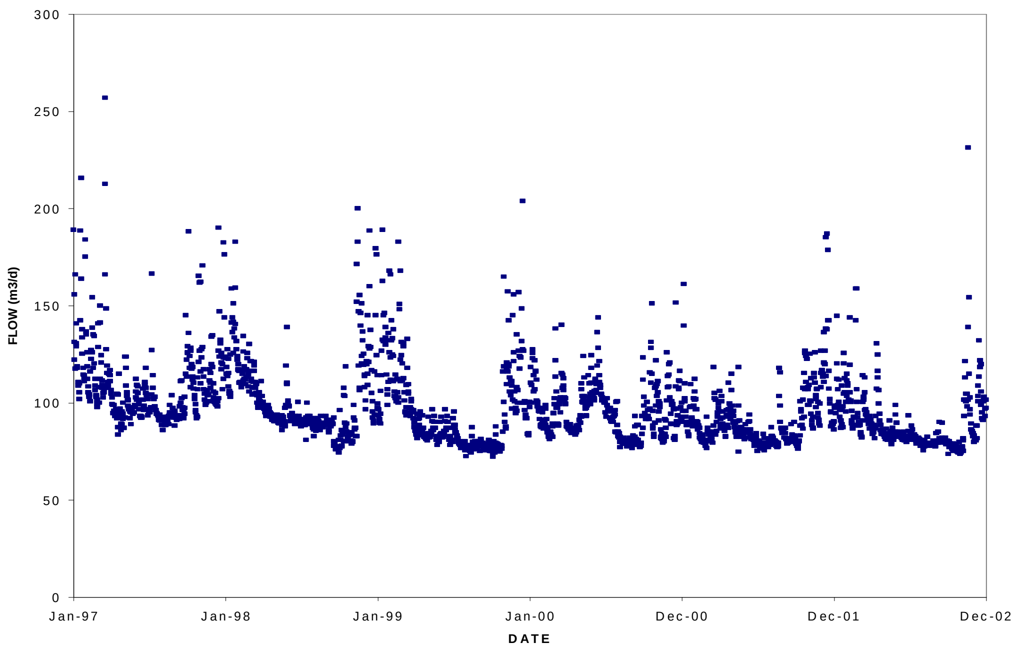


FIGURE 5.25
LGWWTP EFFLUENT BOD CONCENTRATION and LOAD (1997 ~ 2003)

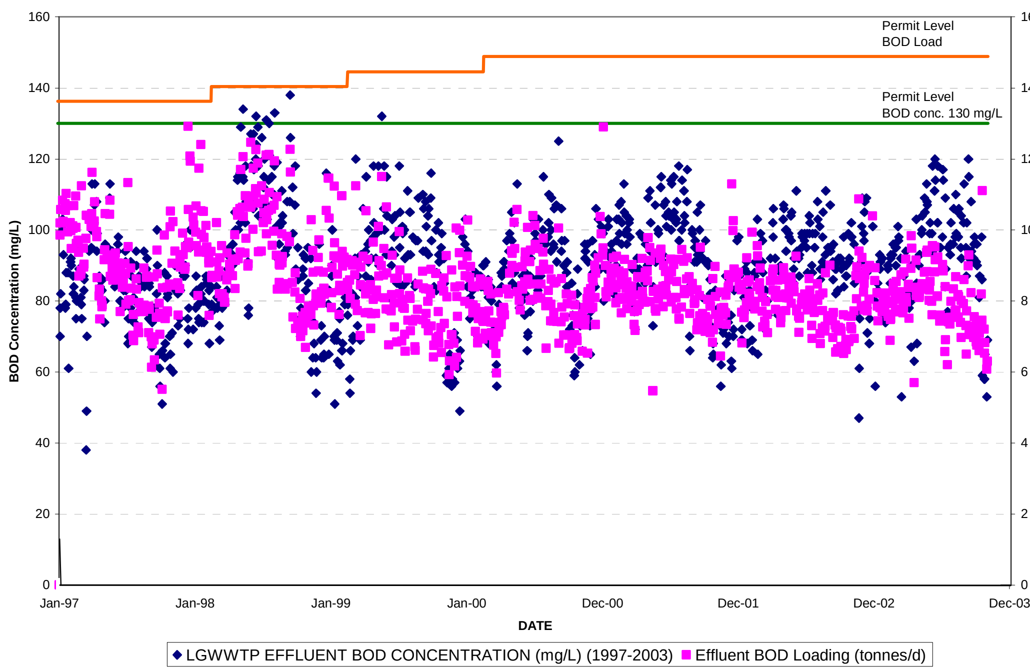


FIGURE 5.26
LGWWTP EFFLUENT TSS CONCENTRATION AND LOAD (1997 ~ 2003)

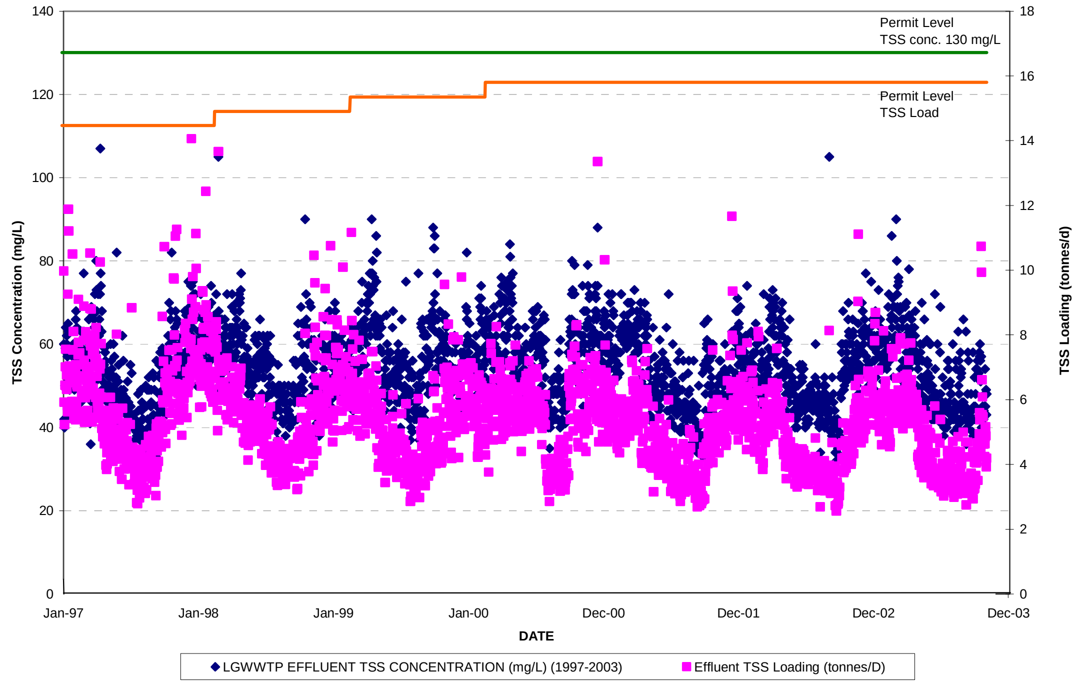


TABLE 5.18
LGWWTP EFFLUENT TOXICITY TEST RESULTS (1997 TO 2003)

Year/ Month	96 hour LC_{50} (%V/V)												No. of Sample Failed Toxicity Test
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
1997	>100	>100	>100	>100	>100	>100	66	91	>100	>100	>100	>100	2 out of 13
1998	>100	>100	>100	>100	72	>100	81	>100	>100	>100	>100	>100	2 out of 12
1999	>100	>100	>100	>100	36,47,68	>100	69,58,61	81	>100	95	>100	>100	8 out of 16
2000	>100	>100	>100	>100	>100	>100	100, 38	>100	>100, 63, 77,50	>100	>100	<100	4 out of 16
2001	>100	>100	49	>100, 64	>100	96, 58	61	97	72, 95	>100	>100	>100, 85	9 out of 18
2002	>100	>100	>100	>100	75	83, 75, 79	62, >100	59, 60, 77	>100	61, 56, 66	>100, 77, 75, 69	<100	14 out of 31
2003	<100	98, >100	>100	>100	56, 60								3 out of 10

6 ALTERNATIVES TO END OF PIPE TREATMENT FOR IONA ISLAND

6.1 GENERAL

Rather than treating the wastewater in the plant (end-of-pipe treatment), it may be possible to treat the sewage in the collection sewer system en route to the treatment plant (in-pipe treatment), in order to reduce the contaminant loads or modify contaminant constituents. Generally for the IIWWTP serving the Vancouver Sewer Area (VSA), sewage is collected by the 8th Avenue, English Bay, and North Arm interceptor sewers, and conveyed by the Highbury trunk sewer southward, entering the treatment plant by three 1676 mm diameter Fraser River crossing pipes. The approximate hydraulic retention time (HRT) in these major sewer trunks during the dry weather low flow condition can be as long as two hours, which is equivalent to the residence time in the primary sedimentation tank in the plant.

Non-compliance of effluent quality at IIWWTP usually occurs during dry weather, The option of dry weather pre-treatment in the sewer trunk system (i.e. the Highbury sewer trunk) was considered to change sewage characteristics (e.g. prevent soluble BOD production and microorganism growth), or reduce organic loads (additional BOD removal) entering IIWWTP. If such treatment were successful, then it would not be necessary to expand the existing treatment facility. The following approaches would be explored:

- Inhibit soluble BOD generation and microorganism growth by chemical treatment, or by the removal of soluble BOD by the use of biological degradation,
- Modify the physical configuration of the sewer in order to increase or decrease the hydraulic retention time.

6.2 OPTIONS FOR IN-PIPE TREATMENT

Chemical treatment, usually chlorination, has been used for corrosion, odour, and slime growth controls in sewer systems. Chlorination is applied to inhibit the growth of sulphur reducing bacteria under anaerobic conditions, or biological degradation (e.g. hydrolysis) of particulate BOD into soluble BOD in the suspension and sediment phases.

Typical chlorination dosage ranges from 1 mg/L to 10 mg/L, depending on the sewage characteristics, contact time requirement, and residence time in the sewer system. Sodium hypochlorite solution can be used to eliminate the storage of gaseous chlorine. Hydrogen peroxide solution is also an option for microorganism inhibition as well as organic oxidation (BOD removal).

Chemical injection point(s), with proper chemical monitoring and dosage control can be located at the upstream and/or halfway points of the major sewer trunks. Sufficient chemical reaction time and adequate mixing should be provided in the manholes, contact basins, or in-pipe reactors. The appropriate chemical dosage can be determined by pilot scale tests. Tentatively, chemical addition will be required only during the low flow conditions (e.g. during dry weather with long residence time in the sewer trunks).

In contrast, fixed film or suspended microorganisms can be used to achieve biological removal of organic matter in sewer trunks. The sewer trunks can be operated as a bioreactor with suspended and fixed growth microorganism just like any other activated sludge process to achieve organic removal. Usually, the DO concentration is the limiting factor in the sewage collection system. By installing an effective aeration system, an adequate DO level can be maintained to sustain aerobic reactions, and the microorganism populations in the suspension and fixed film phases will become the limiting factor. Aeration in the sewer system can also prevent the wastewater from becoming septic and, as a result, associated odours will be eliminated.

In the literature reviewed, one study suggested that approximately 12 % of soluble COD removal can be achieved in a 3.5 km 2000 mm diameter sewer trunk with 0.011% slope and 0.5 m/s flow rate. However, due to limited full-scale operational information similar to the VSA system, pilot scale testing or model simulation (e.g. Mouse Trap®, DHI®) are recommended to assess its feasibility and efficacy.

6.3 ANALYSIS AND RECOMMENDATIONS

It was postulated that toxicity reduction at Iona Island WWTP could be significantly influenced by actions taken in the sewer system. The primary cause of the toxicity problem at Iona has been identified as oxygen depletion occurring during the compliance monitoring toxicity testing. This is due to the presence of readily degradable organics in the primary effluent as well as the action of microorganisms (bacteria and protozoans) present in the primary effluent. GVRD personnel have noted the presence of organisms, similar to activated sludge organisms in the samples sent to the toxicity-testing laboratory at times when the compliance toxicity tests have failed – usually during July and August low flow periods.

Strategies to eliminate and reduce the occurrence of toxicity test failures that have been proposed are:

1. Reduce the source of soluble organics:

    a) By industrial source control.
    b) By aeration of a portion of Highbury Tunnel
    c) By adding oxidizing chemicals such as hydrogen peroxide to the sewer system.

2. Suppress the growth of microorganisms in the sewers:

    a) By adding a strong oxidizing agent such as chlorine to the sewers.
    b) By changing the flow regime in the sewers.

3. Controlling the development of soluble organics in the sewers:

a) By changing the flow regime and eliminating the potential for sludge accumulation.

If such strategies are feasible, the cost may be significantly less than providing interim treatment at the Iona plant.

As part of this study, two activities have provided information on how effective in-sewer control activities might be. The first activity, which was carried out as part of Appendix 2, included field sampling for solids, soluble and total organics, dissolved oxygen, was completed at five locations of the Highbury sewer and in tributary inputs to this major trunk. Modeling of the sewer system model was carried out to calculate flow velocities at average dry weather flow conditions at key locations along the sewer lines.

The second activity, which was carried out as part of Appendix 5, included batch chemically enhanced, and primary and biological treatment. This was undertaken in August of 2003 during the dry weather flow period treating raw and primary effluent at both Iona and Lions Gate Wastewater Treatment Plants. Standard LC_{50} toxicity testing as well as testing for total and soluble BOD_5, TSS, and surfactants (methylene blue active substances, MBAS) was carried out.

The field sampling and testing in the sewer system were limited in nature but did show the following:

- Little trending in soluble organics or TSS occurred in the major trunk sewer from upstream to downstream sections of the Highbury tunnel other than an expected increase in organic, and solid load, consistent with increased inputs along the trunk sewer.
- Flow velocity calculations from the model indicated that, even during average low flow conditions, and the velocities in the main trunks did not decrease to levels where organics would settle out into the invert of the sewers.
- Throughout the trunk sewer system sampled, the dissolved oxygen levels were essentially zero.

From this information, it appears that there are microorganisms at work in the sewers that are utilizing the available dissolved oxygen but these are not significantly reducing the organic loading.

The six sets of small-scale treatment batch tests provided good information on the extent of organics and surfactants removal that has to be achieved to obtain a significant reduction in toxicity. To reduce the frequency of occurrence of acute toxicity test failure, at least 100% chemically enhanced primary or 50% biological treatment (100% of load receiving primary settling plus 50% of ADWF biological treatment) has to be carried out. The required extent of soluble organics removal needed appears to be 52 to 77%. This is a very significant reduction in organics.

From these tests the following conclusions can be drawn:

- Reducing the industrial organics load would not be feasible because at the Iona Island plant, the total industrial load represents only about 15% of the total BOD load to the plant. Forcing pretreatment on industry by a by-law change would not be successful because a 50 to 77% reduction in mass loading would not be physically possible through industrial load reduction alone.

- Similarly, to achieve a mass reduction in soluble BOD in the range of 50 to 77% would essentially mean converting a portion of the sewer system into a biological or chemical treatment facility. The addition of chemical oxidants such as hydrogen peroxide, potassium permanganate, or ferric salts could not achieve the required level of organic destruction at a reasonable operating cost. Creating an in-sewer, tubular reactor biological treatment system would require the equivalent, or greater, capital cost than partial biological (50% ADWF) treatment at the Iona plant. We are not aware of a major application of these techniques in North America. Transport of the biological solids generated by such an in-sewer system would also be problematic.

- In-sewer treatment is not a feasible option.

7 INTERIM TREATMENT ALTERNATIVES

7.1 PHYSICAL-CHEMICAL PROCESSES

7.1.1 Chemically Enhanced Primary

Process Description

The performance of primary sedimentation can be improved by the addition of chemicals to promote coagulation and flocculation of suspended and colloidal solids, as well as precipitation of some dissolved compounds such as phosphorus. Normally a metal salt (alum, iron, or calcium) is the primary chemical added. Small amounts of patented polymers are often added as well, to enhance performance. Chemical injection is normally followed by turbulent mixing, and then a short period of gentle mixing to promote flocculation before the treated liquid enters the sedimentation tanks. A process schematic and summarized technical facts are presented in Figure 7.1.

Optimum chemical doses depend on the characteristics of the wastewater, and onsite testing is generally recommended. For the Iona and Lions Gate WWTPs, limited onsite testing has shown that a dose of about 70 to 80 mg/L alum or ferric chloride combined with 0.5 to 1 mg/L anionic polymer may be the optimum dose for removal of BOD and TSS (see Discharge Requirement/Effluent Quality below).

Proven Technology

CEP is a well-established process and is commonly in use in North America and elsewhere in the world.

Discharge Requirement/Effluent Quality

The performance of CEP is subject to the amount of chemical addition and the overflow rate of the primary sedimentation tanks. Metcalf & Eddy, 2002 recommend surface overflow rates (SOR) for CEP with alum or iron addition of 30-70 $m^3/m^2/d$ at average flow, and 80 $m^3/m^2/d$ at peak hourly flow; this compares to 30-50 $m^3/m^2/d$ at average flow and 100 $m^3/m^2/d$ at peak hourly flow recommended for primary sedimentation without chemical addition followed by secondary treatment. According to Metcalf & Eddy (2002), with chemical addition it is possible to remove 80% to 90% TSS (including some colloidal particles), 50% to 80% BOD, and 80% to 90% of bacteria; comparable removals for well-designed primary sedimentation tanks without chemical addition are 50% to 70% TSS, 25% to 40% BOD, and 25% to 75% of bacteria.

Limited pilot scale testing at Iona WWTP has shown that CEP can meet the interim effluent requirements of 130 mg/L for BOD and 100 mg/L TSS for typical domestic wastewater at a chemical dose of about 75-mg/L ferric chloride and 1 mg/L anionic polymer with SORs in the range 60-80 $m^3/m^2/d$. This level of chemical addition reportedly improved removal of BOD and TSS by 35%-60% and 65%-95%, respectively at Iona. It was noted that the discharge of industrial waste containing high amounts of soluble BOD might cause discharge limits to be exceeded regardless of chemical addition (Associated, February 1999 and April 1999). Others have recommended lower chemical doses of 10-30 mg/L ferric chloride and 0.1-0.3 mg/L polymer based on bench-scale and pilot- scale testing at Iona; this level of chemical addition improved removal of BOD and TSS by 7%-10% and 15%-25%, respectively (CH2MHill, 1997/98). Stress testing of the primary tanks at Iona in 1996 (no chemical addition) showed that there was no deterioration in effluent quality when the SOR was increased to 140 $m^3/m^2/d$ (CH2M Gore & Storrie, 1996).

Bench-scale testing at Lions Gate WWTP showed that both ferric chloride and alum at a dose of 75 mg/L produced BOD and TSS removal efficiencies of 80% and 85%, respectively (effluent concentrations 40 mg/L BOD and 30 mg/L TSS); the addition of anionic polymer at a dose of 0.25 mg/L significantly increased floc size and improved settling rate (Associated, 1988). Stress testing of the primary tanks at Lions Gate in 1996 (no chemical addition) showed that TSS removal decreased from 80% at an SOR of 70 $m^3/m^2/d$ to 50% at an SOR of 180 $m^3/m^2/d$, but was still within the effluent limit of 130 mg/L. It was estimated that chemical addition could reduce effluent BOD by about 30 mg/L during dry weather flows (CH2M Gore & Storrie, 1996).

Bench-scale testing at Lions Gate has indicated that about 50% removal of surfactants measured as methylene blue active substances (MBAS) is possible using a dose of about 30-50 mg/L alum, which was found to be more effective than ferric chloride (CH2MHill, 2002).

Additional small-scale testing is currently underway at Iona and Lions Gate to evaluate the effectiveness of CEP for removal of TSS, BOD, and acute toxicity, which has been attributed to oxygen demand at Iona and to MBAS at Lions Gate.

Reliability

CEP is generally a simple and reliable process, which is effective immediately upon start-up. Changes in the characteristics of influent sewage would probably not significantly affect the effluent quality within a typical range. Chemical dosing may be flow paced to address fluctuating influent flows. Automatic feedback control of chemical dosing based on real time continuous TSS measurements may also be possible. Chemical addition can be operated continuously or on a peak hour only basis, since it requires minimum response time and process acclimation.

Site Suitability

Retrofitting CEP to existing primary treatment at both Iona and Lions Gate would improve removal of suspended solids and BOD without additional tankage to serve future peak flows. No extra footprint would be required to facilitate this retrofit except the chemical storage area.

FIGURE 7.1
CHEMICALLY ENHANCED PRIMARY TREATMENT (CEP)

Process Description

Chemically Enhanced Primary Treatment is the process by which chemicals, typically metal salts and/or flocculent (polymers are added to primary sedimentation basins. The chemicals cause the suspended particles to clump together via the processes of coagulation and flocculation. Removal of suspended solids is significantly enhanced compared to conventional primary settling.

Design Criteria

Overflow rate:
- 30-70 $m^3/m^2/h$ at average flow
- 100 $m^3/m^2/h$ at peak hourly flow

Chemical Dose (typical at Iona Island & Lions Gate):
- 70 to 80 mg/L alum or ferric chloride
- 0.5 to 1 mg/L anionic polymer

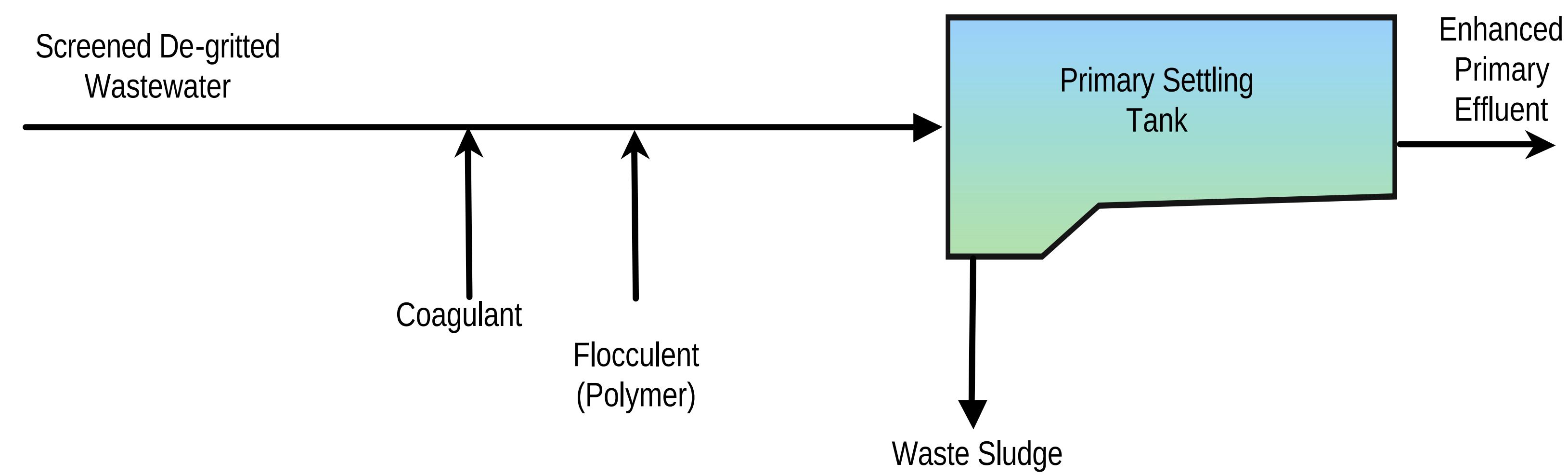
Expected Performance

Parameter	Effluent (Percent Removal)
BOD_5 mg/L	70 (50 - 80%)
TSS mg/L	40 (80 – 90%)

Plant Footprint

Would not be expanded beyond existing facilities except for chemical storage

Schematics



Advantages

- Treatment efficiency is enhanced because particles aggregate and flocs settle faster
- Low capital cost
- Simple to retrofit existing primary processes
- Smaller footprint than conventional primary treatment
- Can meet interim LWMP goals for BOD/TSS

Disadvantages

- Bench-scale and pilot plant tests must be done on site to determine effective coagulant chemicals and dosages
- Chemical addition significantly increases the amount of waste solids
- High cost for chemicals
- Will not consistently remove MBAS toxicity or ammonia toxicity

Comments

- Could be applied at Lions Gate and Iona Island to ensure permit compliance

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7.1.2 CEP with Lamella Retrofit to Existing Primaries

Process Description

Chemically enhanced primary (CEP) treatment with lamella retrofit is a process that involves the use of chemical precipitation and lamella plate settling to enhance solids capture. The process schematic and technical information are summarized in Figure 7.2. Liquid alum or ferric salts will be applied to the screened and degritted wastewater flows at 150 mg/L and 75 mg/L respectively. Using ferric chloride, a dosage of 50 ~ 75 mg/L is required as indicated in the pilot-scale study carried out by Associated Engineering in 1999. Based on literature review, more alum dosage is required to achieve the same removal, resulting in approximately 100 ~ 150 mg/L of alum. Anionic polymer is also added at approximately 1.0 mg/L to increase the settleability and capture of TSS. Existing pre-aeration tanks will serve as coagulation/flocculation tanks, which provide optimum growth of chemical flocs.

The plates, inclined at an angle of 45° to 60° from the horizontal, will be closely spaced and cover approximately 80% of the sedimentation tank area. As the flocculated wastewater enters the primary sedimentation tank and rises to the lamella plates (e.g. counter-current application), effluent will pass through the plates while solids will settle and slide down the inclined plates and be collected at the bottom of the tank as waste sludge.

Inclined lamella plates are intended to enhance solids separation and result in a higher removal efficiency of suspended solids than CEP alone, especially during high loading events. About 10 to 15% increase in sludge production will be expected compared to the CEP process. The capacities of sludge collection pumping, thickening and stabilization (digestion) facilities currently serving the primary plants will have to be upgraded to accommodate the extra solids removed and chemical sludge produced. The process may require more intensive labor to maintain the lamella plates and has higher capital cost than CEP. Experience gained in the City of Laval and Longueil, Quebec suggested that frequent cleaning of the lamella is required. In Longueil, its lamella plates are cleaned using a diffused air system, which is activated for once every four days. In Laval, the water level in its clarifier is lowered and lamella plates are hosed down every two weeks.

Proven Technology

Chemically enhanced lamella plate sedimentation has been used worldwide in many municipal and industrial wastewater applications. There are few examples of lamella retrofit in existing primary sedimentation tanks. However, there are several examples of primary sedimentation tanks that were specifically designed to utilize lamella. One such example is the City of Longueil CEPS plant near Montreal. The ADWF for its CEPS Plant is 300 Ml/D. This plant has eight primary sedimentation tanks with lamella (33 m x 4 m) preceded by four flocculation chambers (9.4 m x 29 m). TSS removal of 85% is achieved using an alum dosage of 40 mg/L and polymer aids.

Discharge Requirement/Effluent Quality

The expected effluent quality is 80 mg/L of BOD_5 and 50 mg/L of TSS, based on removal efficiencies of 60% of BOD_5 and 80% of TSS respectively. The removal of soluble BOD_5 will be limited with this application. The performance is subject to the quantity of chemical addition and the number of lamella plates. Pilot-scale tests are recommended to determine the optimum chemical dosage and lamella plate requirement. MBAS removal will be limited to a maximum of 60% at high chemical doses (>100 mg/L alum).

Reliability

Changes in influent sewage characteristics would not significantly affect the effluent quality within a typical range. Chemical dosage can be adjusted dynamically to accommodate flow and load variances. Chemical addition can also be operated continuously or at peak hours only, requiring minimum response time and process acclimation. Cleaning of the lamella plates can be arranged to coincide with low flow periods.

Site Suitability

To serve future peak flows, retrofitting CEP with modular lamella to existing primary sedimentation tanks will improve suspended solid removal efficiency without the need for additional tankage. Subject to the design conditions, only some of the primary tanks will require the lamella installation. No extra footprint is required for this retrofit except for the chemical storage area. Modular lamella plates will be installed in existing primary sedimentation basins. The sludge and scum scraper systems should be modified to accommodate the lamella installation. However, the sedimentation tanks are shallow (2.7 ~ 2.9 m water depth), and so the length and angle of the lamella plates should be designed to compensate for this limitation. Either that, or the tank depth needs to be increased.

As space is a constraint at LGWWTP, CEP with lamella will improve the solids removal capacity in the existing sedimentation tanks, thus allowing the remaining units to be used for biological treatment. The tank depth at LGWWTP can be increased, if necessary, to provide sufficient space for the lamella installation and to increase the hydraulic retention time. The CEP and lamella can also be applied at IIWWTP to provide extra solids removal capacity, but it will require significant foundation capacity improvement to allow for the increase in tank depth.

FIGURE 7.2
CEP WITH LAMELLA RETROFIT TO EXISTING PRIMARIES

Process Description

Inclined plates set at a maximum angle of 60º will be retrofitted into the existing primary settling tanks. Liquid alum or ferric chloride will be applied to the screened and degritted wastewater flows at application rates of 150 mg/L and 75 mg/L, respectively. To increase the settleability or capture of TSS, polymer will also be added at approximately 0.5~1.0 mg/L. Flocculation tanks will be provided following chemical addition to provide optimum growth of chemical floc (use existing pre-aeration basins). As part of this process the sludge collection pumping, thickening and digestion facilities currently serving the primary plants would have to be increased in capacity to accommodate the extra solids captured and chemical sludge generated.

Design Criteria

SOR: 10 m/hr
Depth of plates: 1.0 - 1.5 m
Max. angle of plate: 60º
% of settling tanks covered by plates: ~ 80%
Primary sedimentation tank depth: 5.5 - 6.5 m
Flocculation tank HRT: 10 – 15 min
Paddle mixer energy: 3 - 5 kW/1000 m^3
Air agitation: 450 - 900 m^3 air/m^3 reactor
Depth of flocculation tank: 3 - 5 m

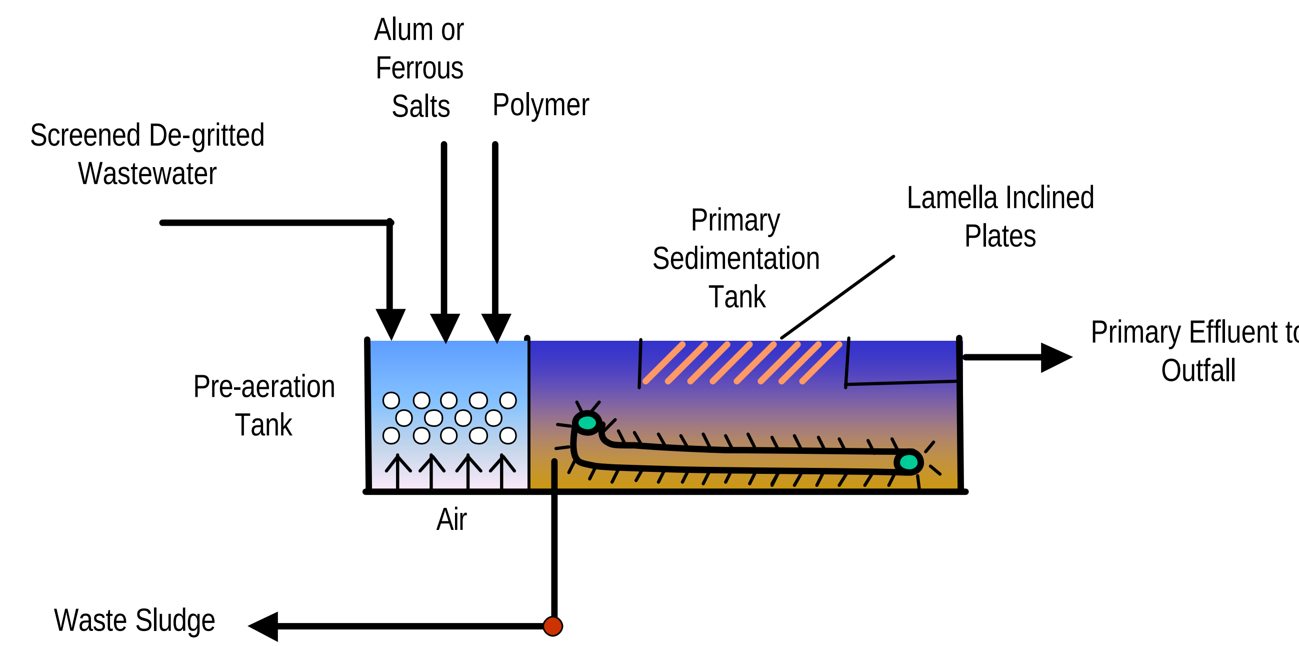
Expected Performance

Parameter	CEP + Lamella Effluent
BOD_5 mg/L	60
TSS mg/L	30

Plant Footprint

No additional land imposed upon existing primary facilities except chemical storage area

Schematics



Advantages

- Only some of primary tanks need equipping
- No extra footprint required
- Thicker sludge than CEP alone
- Better treatment at higher capacity flows
- No additional tankage to serve future flows at IIWWTP or LGWWTP
- Robust control and operational flexibility

Disadvantages

- Slightly more sludge production than CEP alone (~ 5%), but 100 to 150 % more than the existing primary treatment
- More labour intensive – keep lamella clean
- Not frequently applied in north America
- Higher capital cost than CEP
- Modification of scum and sludge scraper system is required

Comment

- Could be applied at Lions Gate to allow use of remaining primary clarifier area as a base for biological treatment
- The depth of sedimentation tanks needs to be increased from 2.7 - 2.9 m to 5.5 - 6.5 m, which may impose difficulty at IIWWTP

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7.1.3 DensaDeg

Process Description

DensaDeg® is a proprietary process that combines mixing, internal and external solids recirculation, sludge thickening and lamella clarification in two adjoining vessels. A process schematics and technical information are summarized in Figure 7.3. The DensaDeg® clarifier incorporates three process zones: the reactor zone, the presettling/thickener zone and the clarification zone. In the reactor zone, the influent water is combined with chemicals and solids that have been recirculated from the presettling/thickener zone. A draft tube and turbine are the key components of the reactor zone. As the raw water, chemicals and recirculated solids flow upward together through the draft tube, a flocculated mixture is formed. Exiting the draft tube, the flocculated mixture flows downward. A significant amount of the flocculated slurry re-enters the draft tube.

A portion of the slurry passes to the next vessel into the presettling/thickening zone. Dynamic separation of the solids and supernatant occurs in this zone. As the slurry moves downward though the presettling zone, to a point near the bottom of the vessel, is has to make a 180° turn beneath the baffle that vertically divides the vessel. A large proportion of the solids are deposited at this point. Openings in the baffle are located at the bottom. The supernatant moves upward in the second compartment. Lamella tubes located near the surface provide for additional removal of solids. A slow moving rake located in the bottom of the second vessel removes the solids. Thickened sludge is periodically pumped down from the bottom of the thickener directly into a digester. Some of the solids are recirculated back to the reactor zone where they are mixed with the raw influent and the chemicals.

Proven Technology

The DensaDeg® process has been in operation since 1998 at the La Pinière wastewater treatment plant in Laval, QC. This process was selected because of its small footprint. The flow in this plant can vary between 175,000 and 600,000 m^3/day. There are 6 DensaDeg® modules each measuring 17 m x 7 m at the plant. Pretreatment includes a 12 mm bar screen and grit removal. No biological treatment is provided following the DensaDeg® process. Disinfection is provided using UV prior to river discharge.

The facility is designed to handle a maximum flow of 700,000 m^3/day. Discussions with the operating staff have indicated that an alum dosage of 60 mg/L and a polymer dosage of 0.25 mg/L are used. The City appears to be generally satisfied with the DensaDeg® process except that cleaning of the lamella tubes necessitates taking the tank out of service and lowering the water level in order to clean the tubes with a hose. One option for improving the cleaning operation would be to add coarse bubble diffusers below the lamella.

Discharge Requirements/Effluent Quality

The effluent quality of the La Pinière plant during dry weather (less than 250,000 m^3/day) is generally good at 15 ~ 20 mg/L for TSS and 20 ~ 25 mg/L BOD_5. The influent raw wastewater has a TSS of 140 ~ 150 mg/L and a BOD_5 of 75 ~ 80 mg/L.

Reliability

Changes of influent sewage characteristics would not significantly affect the effluent quality within a typical range. Chemical dosage can be adjusted dynamically to accommodate flow and load variances. Chemical addition can also be operated continuously or at peak hours only, requiring minimum response time and process acclimation. Cleaning of the lamella plates can be arranged during low flow periods.

Site Suitability

The DensaDeg® process requires the construction of dedicated concrete tanks sized for the specific requirements of the equipment. This process cannot be retrofitted inside the existing primary sedimentation tanks. Because of its small footprint, this process could be installed at Iona Island and Lions Gate to replace the existing primary sedimentation tanks or to add capacity.

DENSADEG®

Process Description

Densadeg® is a patented high-rate solids contact clarifier that is reported to achieve high hydraulic loading rates and treatment efficiencies by combining optimized flocculation, internal and external sludge recirculation, and tube settling in two conjoined vessels. In the reactor zone, recycled chemically conditioned solids are used to form microfloc particles in the incoming wastewater. A metal coagulant and polymer are added. Recirculation of solids accelerates flocculation and forms dense particles. The flow from the reaction zone enters a lamella plate clarifier. The integration of operations within each vessel has facilitated a significant reduction in the size of the treatment unit.

Design Criteria

- Rise rate up to 25 m/h for most metal salt coagulation processes
- The chemical dosage and detailed criteria need to be custom designed by the suppliers
-

Expected Performance

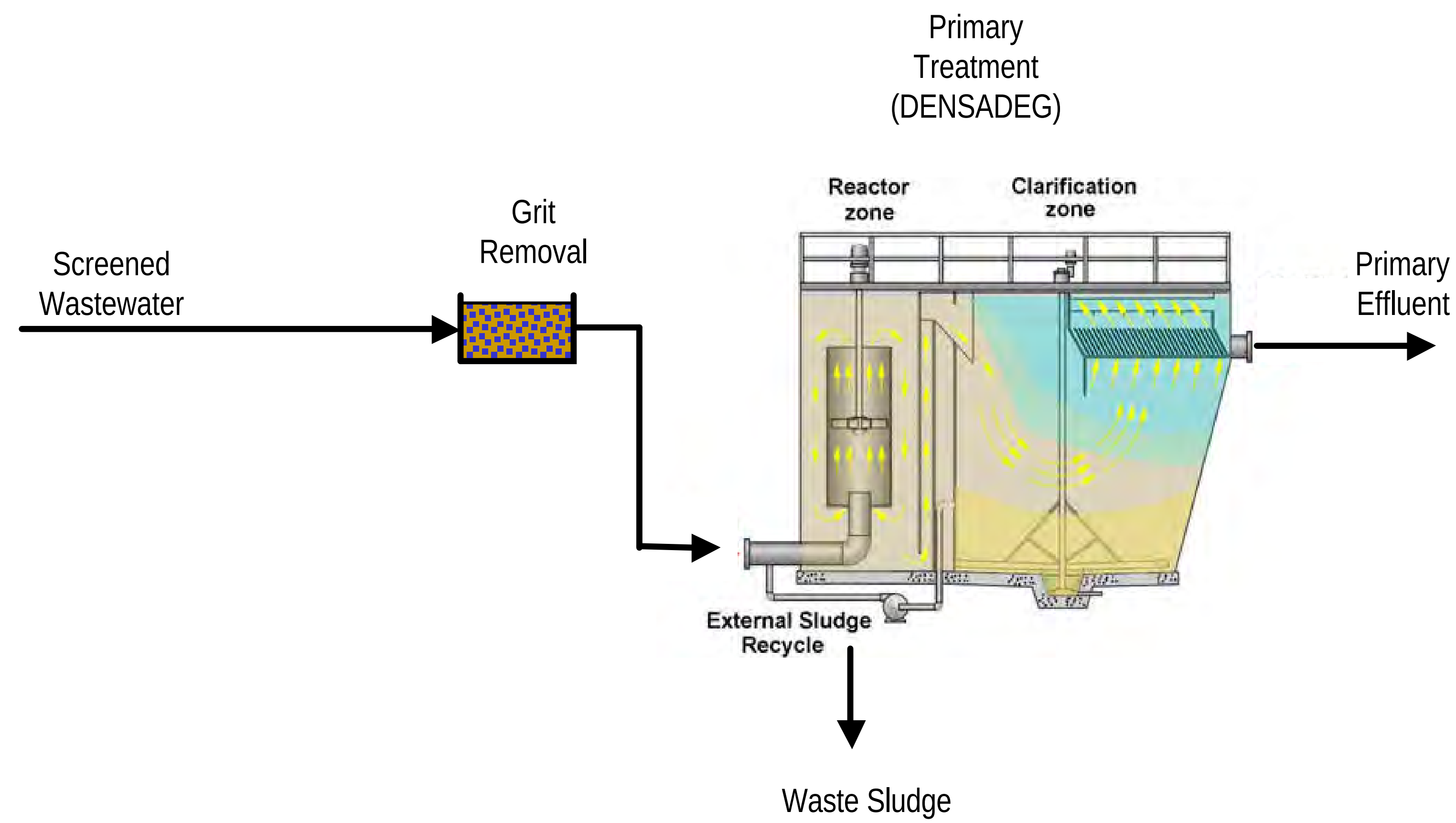
The performance of is subject to chemical dosage, wastewater composition, effective contact time, pH, and temperature.

Parameter	Treated Effluent
BOD_5 mg/L	70
TSS mg/L	30

Plant Footprint

IIWWTP: Approximately 2,800 m^2 is required to treat 100% of the flow
LGWWTP: Approximately 700 m^2 is required to treat 100% of the flow

Schematics



Advantages

- Small footprint
- Dense waste sludge
- Internal and external sludge recirculation and high reactor solids concentration reduces startup time and accelerates treatment rates
- Rapid sludge settling

Disadvantages

- Cost of chemicals
- Limited performance history

Comments

- Small footprint, potential option for Lions Gate WWTP

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7.1.4 Ballasted Flocculation Retrofit

Process Description

The ballasted flocculation process is a proprietary technology provided by US Filter/John Meunier under the trade name Actiflo®. A process schematic and technical information are illustrated in Figure 7.4. The Actiflo® high rate clarification process is designed to remove solids through enhanced chemical flocculation and gravity settling. The system is unique in its use of sand (silica microsand) to assist with chemically aided floc formation. In addition to providing a nucleus for floc formation, the sand increases the mass of the settling flocs and results in ballasted flocculation. The Actiflo® process can be operated at 80 ~ 120 m/hr of surface overflow rate, which is approximately ten times higher than the conventional sedimentation basin overflow rate.

Alum or ferrous salts (coagulants) are injected into the screened and degritted wastewater stream prior to entering the injection basins where sand is added and mixing occurs. The coagulation and flocculation process begins. The sand coagulant mixture enters the maturation tanks where slow mixing allows the attachment of flocs to the microsand. Settling of ballasted floc occurs in the existing primary sedimentation tanks in which modular lamella plates are installed. Supernatant is collected at the effluent launders and settled sludge with microsand is pumped from the bottom of the tanks and through a hydrocyclone, which is used to separate the sand from the floc. The separated sand is recycled into the process while the sludge is wasted to the gravity thickener or returned to the influent raw wastewater stream for co-thickening.

Due to the chemical addition and high rate solids removal, more sludge will be produced and associated facility upgrades are required including sludge collection, sludge thickener, digester and disposal capacity. The hydrocyclone, recycle pumps, and lamella settlers require routine maintenance to sustain optimum operational condition. Microsand losses are expected in the operation and replacement is also necessary.

Proven Technology

Ballasted flocculation process has been used in water and industrial wastewater treatment for high rate solids removal. Sewage treatment with ballasted flocculation is increasing, particularly for primary and combined sewer overflow (CSO) treatments. Full-scale applications can be found in Boisbriand, Quebec, St. Bernard, Louisiana and Port Clinton, Ohio.

Discharge Requirement/Effluent Quality

Expected removal efficiencies, based on a full-scale installation, are up to 60% for BOD_5 and 85% for TSS. The effluent BOD_5 is expected to be 80 mg/L and the TSS 30 mg/L. Operating conditions including chemical dosage, underflow rate, and microsand recirculation can be adjusted based on variations in flow and load. MBAS removal will be limited at the low flocculent dosages required for coagulation.

Reliability

Ballasted flocculation is considered to be a robust process which can be started up within 10 ~ 15 minutes. The chemical dosage, underflow rate, and recirculation rate can be adjusted dynamically to accommodate changes in flow and load. The effluent quality of the ballasted flocculation process is considered to be consistent and reliable.

Site Suitability

The existing tank depth is about 2.7 ~ 2.9 m, but the Actiflo® requires a minimum depth of 6.5 m. The existing sedimentation tanks cannot be retrofitted with the Actiflo® process, unless the tank depth can be increased. Additional real estate will be required if the Actiflo is selected to replace the existing sedimentation tanks. For the interim upgrade, in order to treat 100% of the flow about 2,600 m^2 would be required at IIWWTP, and 650 m^2 at LGWWTP. Using Actiflo® to treat a portion of the flow is also feasible in order to provide additional capacity and enhance effluent quality.

FIGURE 7.4
BALLASTED FLOCCULATION (ACTIFLO®)

Process Description

The ballasted flocculation is marketed by John Meunier/US Filter by the trade name of Actiflo®. Raw wastewater passed through the grit chamber will be discharged to a coagulation basin where a coagulant (such as alum or ferrous salts) is added to destabilized suspended solids. The coagulated water will then be mixed with polymer and microsand in the injection basin. Microsand serves as seed for floc formation and as ballast to increase floc density and settling velocity. The resulting flocs aggregate in the maturation tank and settle in the primary sedimentation tank. The flocculated water pass through the lamella plate clarifier (with inclined plates) and is collected at the top of the sedimentation tank. The microsand and sludge collected at the bottom of the primary sedimentation tank is pumped to a hydrocyclone which separates the sludge from microsand. The microsand is then recycled to the injection basin.

Due to the minimum tank depth requirement of the Actiflo® system, the existing primary sedimentation tank cannot be retrofitted to an ballasted flocculation (BF) system, unless the tank depth can be increased.

Design Criteria

Surface overflow rate: ~ 90 m/hr
HRT: ~30 min

The chemical dosage and detailed criteria need to be custom designed by the pilot-test and suppliers

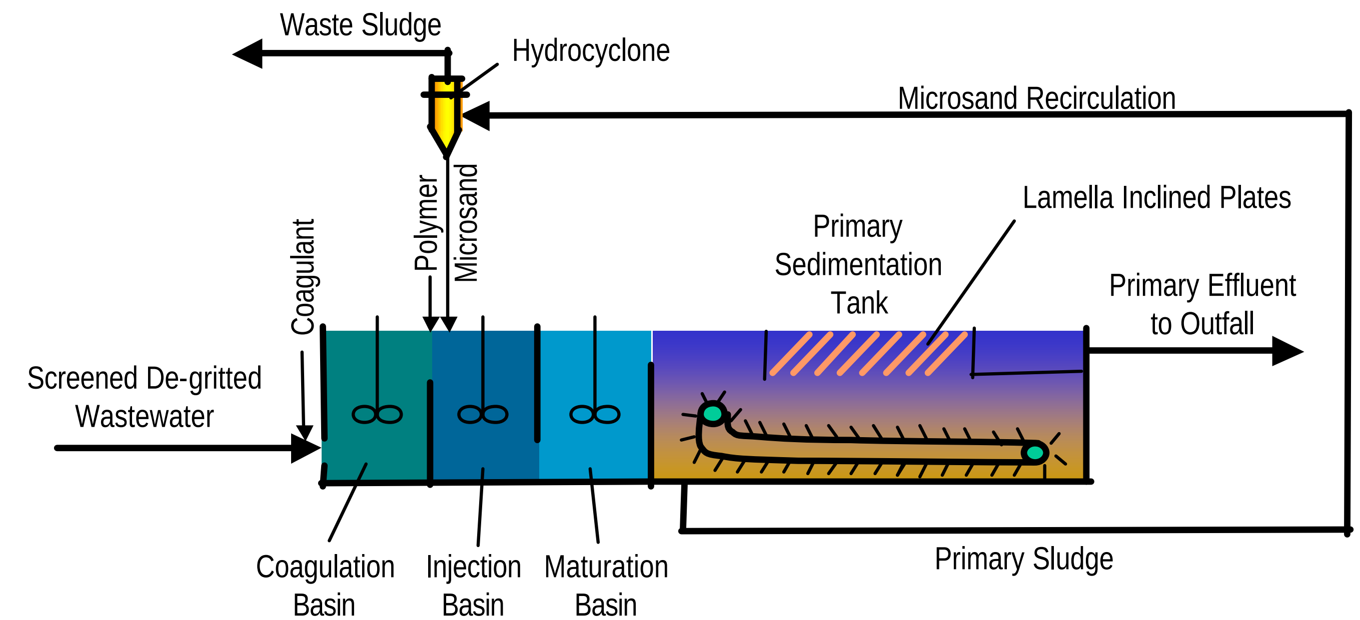
Expected Performance

Parameter	BF Treated Effluent
BOD_5 mg/L	60
TSS mg/L	30

Plant Footprint

IIWWTP: Approximately 2,600 m^2 is required to treat 100% of the flow
LGWWTP: Approximately 650 m^2 is required to treat 100% of the flow

Schematics



Advantages

- More efficient removal efficiency
- Reduced flocculation time, high flow velocities
- Small footprint (than conventional floc/sedimentation/filtration system)

Disadvantages

- Proprietary technology
- More labour intensive to keep Lamella clean
- Microsand replacement
- More sludge produced
- Existing sedimentation tanks cannot be retrofitted due to the depth limitation

Comments

- The Actiflo® system cannot be retrofitted onto existing tankage at both IIWWTP and LGWWTP, unless the sedimentation tanks depth is increased, or new tankage need to be constructed to suit the process need.

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7.2 PARTIAL BIOLOGICAL TREATMENT

7.2.1 General

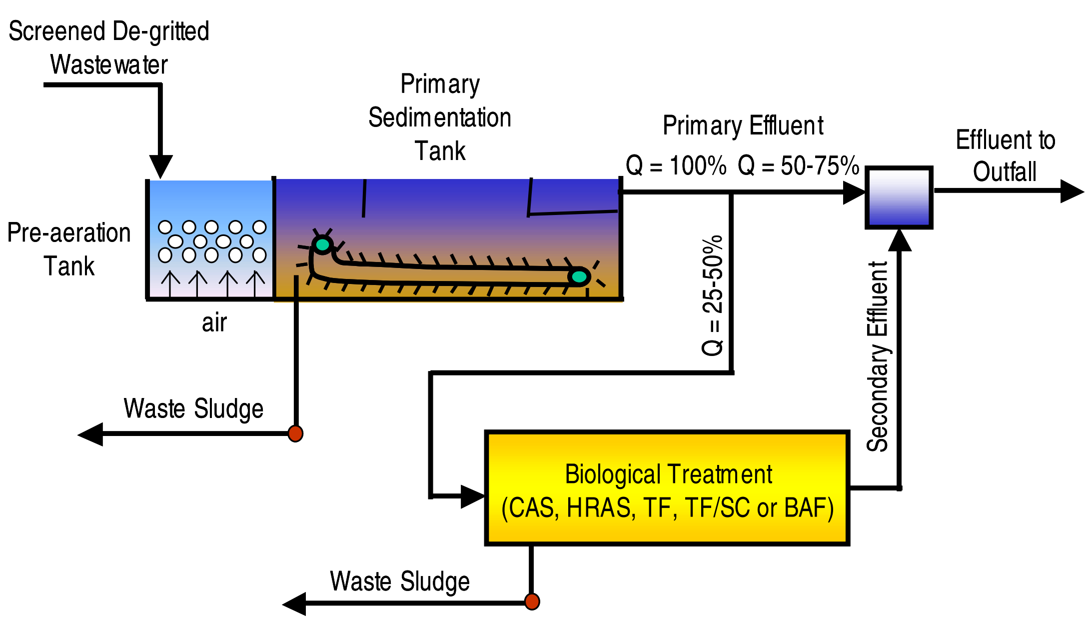
Partial biological treatment consists of providing primary treatment to 100% of the flow using the existing primary sedimentation tanks and treating a portion of the primary effluent using a biological process. The portion of the dry weather flow receiving biological treatment would be 25% to 50%. The portion of the effluent that has received biological treatment would be combined with the portion of the primary effluent prior to discharge to the ocean. The process is shown schematically in Figure 7.5.

The following four biological processes are examined in more detail for partial biological treatment:

- Conventional Activated Sludge (CAS)
- High Rate Activated Sludge (HRAS)
- Roughing Trickling Filter (RTF)
- Biological Aerated Filter (BAF)

The portion of dry weather flow receiving biological treatment depends on how much BOD must be removed.

FIGURE 7.5
PARTIAL BIOLOGICAL TREATMENT - GENERAL



7.2.2 Conventional Activated Sludge (CAS)

Process Description

Conventional activated sludge (CAS) with aerated bioreactors and secondary clarifiers can be added to improve BOD_5 and TSS removal. A process schematic and technical information are summarized in Figure 7.6. Following the primary sedimentation tanks, a portion of primary effluent (25~50%) is aerated in bioreactors in the presence of activated microorganisms (mixed liquor suspended solids, MLSS) for approximately 6 hours. The activated sludge microorganisms utilize organics that remain in the primary effluent as a food source and convert them to biomass, carbon dioxide and water, resulting in BOD_5 removal. Nitrogen levels will also be reduced as a result of biomass synthesis. The MLSS solids are settled out in the secondary clarifiers to achieve TSS reduction.

Approximately 50 to 75% of the MLSS is wasted (waste activated sludge, WAS, also referred as biosolids), thickened and directed to sludge stabilization (anaerobic digesters). The remaining 25 to 50% of the MLSS is recycled as return activated sludge (RAS) to the bioreactor to maintain the solids retention time (SRT) at about 3-6 days, and the MLSS concentration at about 2,500 mg/L. An air supply is required in the bioreactors (usually compressed air supplied from blowers) to meet the oxygen demand of microorganisms in order to keep them in an aerobic condition, to prevent anaerobic fouling and the potential of odour formation.

Compared to the CEP process, the CAS process produces approximately 30% less sludge. Superior sludge treatability is also expected. Higher levels of solids reduction can be achieved during digestion resulting in a lower mass of biosolids to be finally disposed of. Capital costs for the CAS process are higher because of requirements for concrete tankage, the aeration system and pumping equipment, etc. The higher level of operation and maintenance (O/M) (e.g. WAS, RAS, SRT, MLSS, and DO controls) as well as the necessity for certified staff will result in higher operating costs.

Proven Technology

CAS has been widely used worldwide for more than 50 years. CAS is also one of the most cost-effective and common treatment processes for large-scale municipal wastewater application in North America. Substantial design and operating experience has been accumulated providing solid technical support for the application of such treatment. Modified activated sludge processes can be employed in the future as required for nutrient removal in order to meet potentially stringent effluent quality requirements.

Discharge Requirements/Effluent Quality

At a well designed and operated plant treating 25% of the primary effluent, the effluent quality of the combined partial CAS treatment and primary effluent is expected to be approximately 77 mg/L of BOD and 72 mg/L of TSS, respectively.

Reliability

The operation of the activated sludge process requires constant attention in order to maintain the biological culture. Acclimation is also necessary to establish optimum conditions for BOD and TSS removal. System upset may occur due to changes caused by influent shock loading, unfavourable operational conditions, and toxic substances. The loss of microorganisms will result in the deterioration of effluent quality. Preventative measures must be taken to respond appropriately to the upset factors. If action is not taken, recovery following upsets usually takes several days or a week.

Site Suitability

CAS process requires significant land space for the construction of bioreactors and secondary clarifiers. The handling capacity of sludge thickeners and digesters also needs to be increased as required to provide for the increase in sludge production. For the interim upgrade, designed to treat 50% of the flow by the CAS process, approximately 30,000 m^2 and 7,000 m^2 are required at IIWWTP and LGWWTP, respectively. The CAS process can be expanded for the future build out to secondary treatment. At that stage, additional real estate would be required for bioreactors and clarifiers. It appears therefore that there is insufficient space at LGWWTP to accommodate the CAS process.

7.2.3 High Rate Activated Sludge (HRAS)

Process Description

The high rate activated sludge (HRAS) process is a modified version of the conventional activated sludge (CAS). It has higher MLSS concentrations (1,000 - 2,000 mg/L) and a higher volumetric loadings. A process schematic and some technical information are summarized in Figure 7.7. Following the primary sedimentation tanks, a portion of the primary effluent (25-50%) will be diverted to the bioreactors and aerated for approximately 3 - 4.5 hours, or less. The organic matter is utilized by the microorganisms in the bioreactors, and the solids and biomass are settled in the secondary clarifiers. The bioreactor volume of HRAS will be about 30 to 50% smaller than a CAS bioreactor with the same capacity. As a result, less land is required. The food to biomass ratio (F/M) in HRAS is higher than in CAS; the SRT is shorter.

Adequate aeration and mixing is very important for the HRAS operation, as a high MLSS concentration is maintained in the bioreactors. The secondary clarifiers should also be designed to handle the high solids loading. HRAS is often operated without primary sedimentation tanks, therefore, if the existing primary settlers are retained in operation, the bioreactor condition should be properly designed to maintain high rate operation (e.g. F/M ratio, MLSS concentration and SRT etc.). It is also possible to convert the primary tanks into bioreactors if their structural condition is suitable. The production of biosolids will be slightly higher than the CAS process as a result of the shorter SRT.

Proven Technology

HRAS has been widely used in municipal and industrial wastewater treatment as an alternative to CAS. Several installations in North America include Kalispell WWTP, Montana, Twin Fall WWTP, Idaho, and Western Branch WWTP, Maryland.

Discharge Requirement/Effluent Quality

The removal efficiency of HRAS is not as high as that of CAS. Removal efficiencies are usually 10% lower for both BOD_5 and TSS. If 25% of the flow is treated using HRAS, effluent quality parameters are expected to be 81 mg/L of BOD_5 and 75 mg/L of TSS.

Reliability

HRAS requires a similar degree of attention as the CAS process to maintain the stability of the biological cultures for proper treatment. System upsets caused by shock loadings, toxic substances, and abnormal operational conditions are expected to be similar to those experienced in the operation of the CAS process. Shorter acclimation time is anticipated in HRAS compared with CAS due to the shorter SRT. However, some effluent quality deterioration may be experienced for a couple of days or a week.

Site Suitability

As in the case of CAS, HRAS requires a significant capital investment for the bioreactor and secondary clarifiers, as well as for the footprint expansion. For interim upgrades to treat 5% of the flow by the HRAS process, approximately 20,000 m^2 and 5,200 m^2 of additional real estate will be required for IIWWTP and LGWWTP, respectively. The HRAS can be considered as an option for interim upgrades at IIWWTP, and could eventually be extended for the future secondary build-out capacity. However, the site space is limited at LGWWTP for the HRAS interim option and also for the future expansion.

FIGURE 7.6
CONVENTIONAL ACTIVATED SLUDGE (CAS)

Process Description

A portion of primary effluent (25~50%) is aerated in presence of mixed population of activated micro-organisms (activated sludge) for 4~6 hours. The activated sludge organisms utilize organics in wastewater as a food source and convert them to biomass, carbon dioxide and water. The activated sludge is settled out in the final settling tanks. A portion, 50 to 75%, of the activated sludge is wasted, thickened and applied to sludge stabilization (anaerobic digesters). The remainder, about 25 to 50%, is recycled as return activated sludge to the bioreactor to seed the process. Compressed air is applied to the bioreactor to maintain the micro-organisms in an aerobic condition.

In comparison with the CEP process, the CAS process produces less sludge by approximately 30% and superior sludge treatability is expected. Higher level of solids reduction can be achieved in the digestion with the biosolids and less residual is required for final disposal.

Design Criteria

F/M: ~0.4 kg BOD_5 / kg MLSS d
SRT: 3 ~ 6 days
MLSS: 2,000~2,500 mg/L
RAS rate: 25 ~ 50% Q
HRT: 4~6 hours
Tank depth: minimum 4.5 m
Secondary settling tank SOR: 18 m^3/m^2 day (average), 45 m^3/m^2 day (maximum)
Solids loading rate: 120 kg/m^2 day (average), 150 kg/m^2 day (maximum)
Air requirements: 1.2 kg O_2 / kg BOD_5 removed or 7.5m^3 of air / day / m^3 of bioreactor capacity

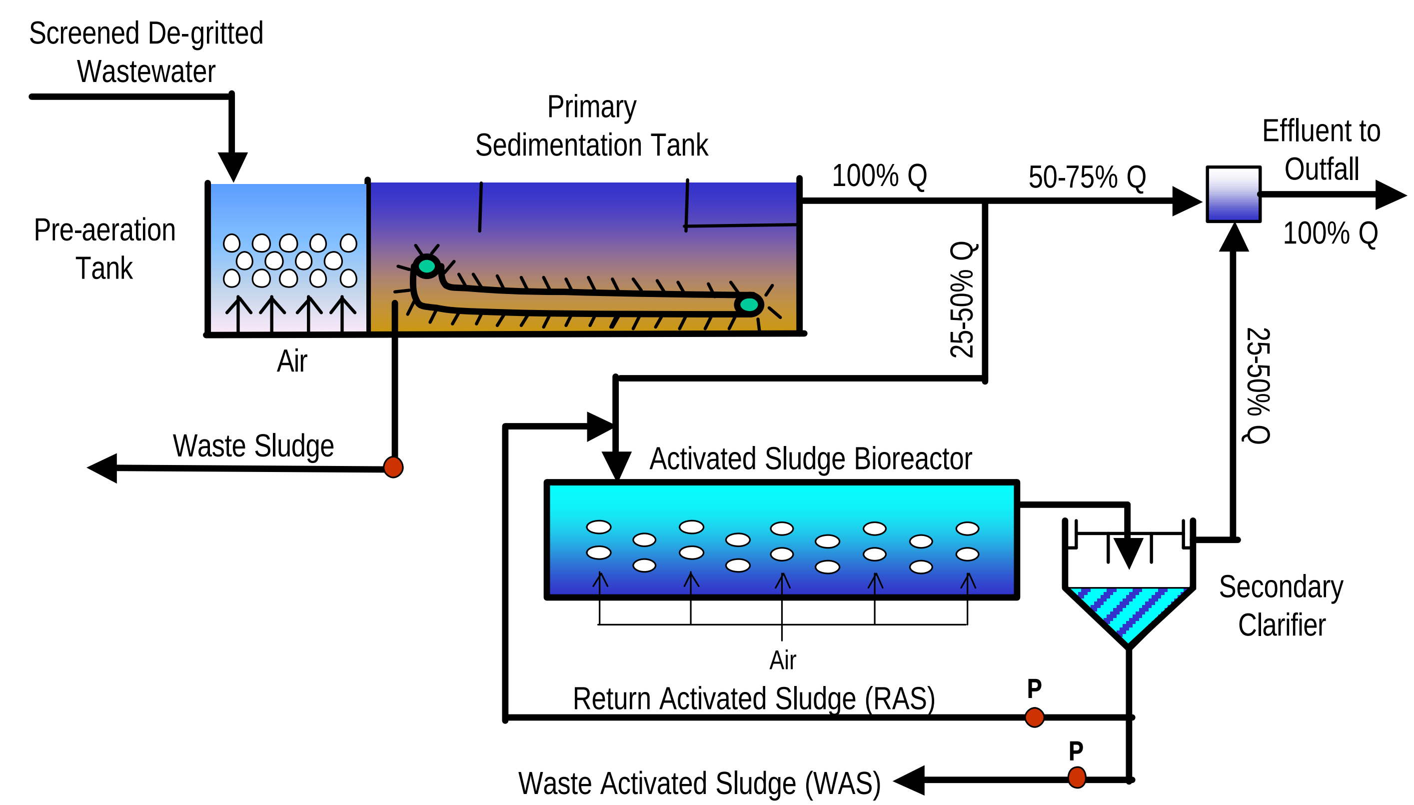
Expected Performance

Parameter	Blended Effluent 25% (50%) of CAS
BOD_5 mg/L	80 (60)
TSS mg/L	75 (55)

Plant Footprint

IIWWTP: Approximately 30,000 m^2 is required to treat 50% of the ADWF
LGWWTP: Approximately 7,000 m^2 is required to treat 50% of the ADWF

Schematics



Advantages

- Proven technology
- Less sludge (~30%) than CEP
- LWMP effluent goals met – BOD, TSS
- Toxicity reduced to LC_{50} of 100% effluent
- Re-use entire process – secondary build-out
- Odour control limited
- Operating costs lower than CEP

Disadvantages

- Significant plant footprint
- High level of O&M
- Additional staff required
- Capital costs higher than CEP
- Different process than AIWWTP and LIWWTP
- Biosolids production increase

Comments

- Good option for IIWWTP
- Poor choice for LGWWTP due to large footprint required

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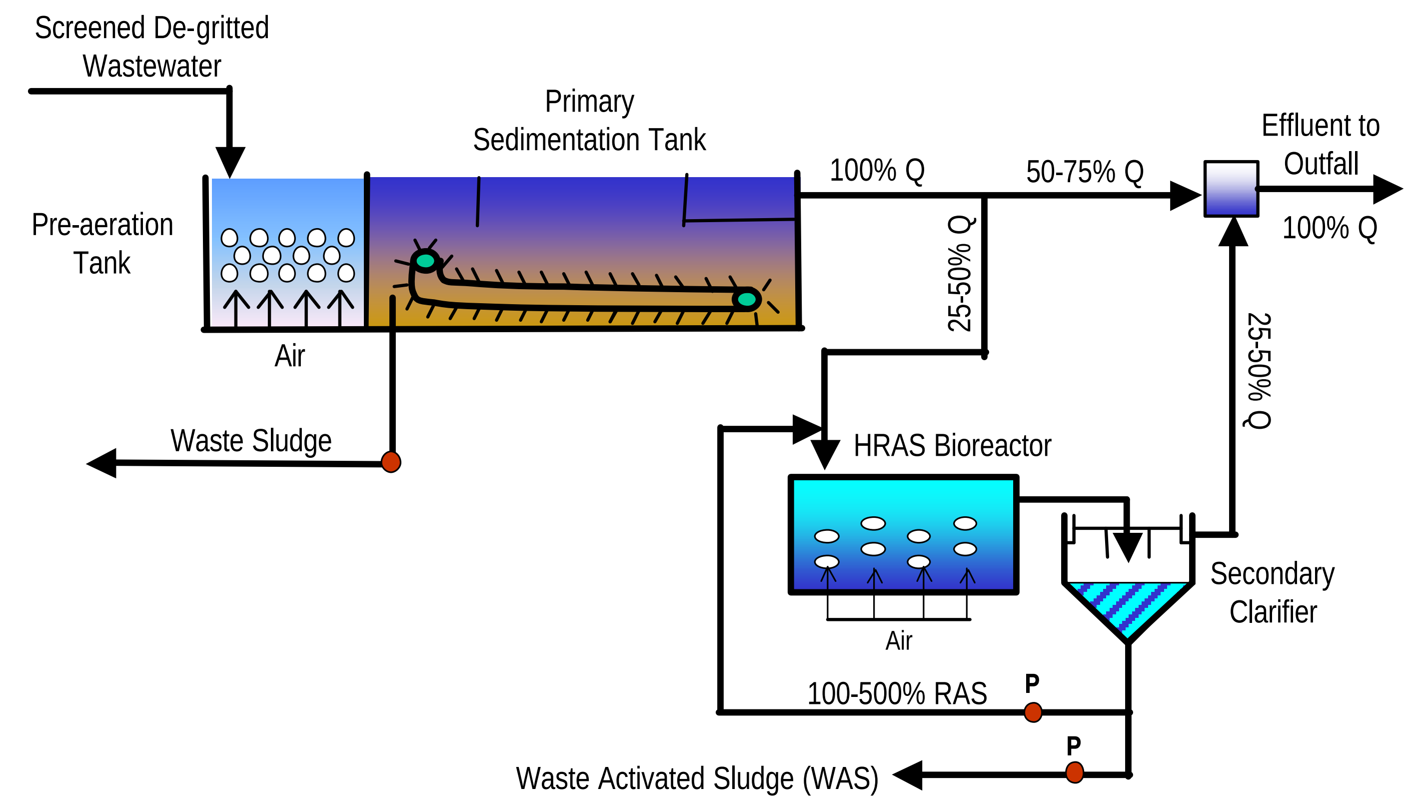
Process Description

This process is a modification of CAS. A portion of primary effluent (25 - 50%) will be diverted to the bioreactors and aerated for approximately 3 - 4.5 hours, or less. The organic matter is utilized by the microorganisms grown in the bioreactors, and the solids and biomass are settled in the sequential secondary clarifiers. In the HRAS process, high MLSS concentrations are combined with high volumetric loadings. Mean cell residence time (MCRT) is longer and F/M ratio is higher than CAS. Removal efficiency is not as good as CAS. Bioreactors for this process are much smaller than CAS (30% to 50% of CAS). HRAS is often applied without primary clarifiers.

It is also possible to convert the primary tanks into bioreactors if their structural conditions are suitable. The production of biosolids will be slightly higher than the CAS process due to shorter SRT.

Schematics



Design Criteria

F/M: 0.4 -1.5 kg BOD_5 / kg MLSS d
MCRT (SRT): 0.75-2 days
MLSS: 1,000 ~ 2,000 mg/L
RAS rate: 100 ~ 500% Q
HRT: 3 ~ 4.5 hours or less
Tank depth: 4.5 ~ 5.0 m
Secondary settling tank SOR: 18 m^3/m^2 day (average), 45 m^3/m^2 day (maximum)
Solids loading rate: 120 kg/m^2 day (average), 150 kg/m^2 day (maximum)
Air requirements: 0.6 kg O_2 / kg BOD_5 removed or 7.5m^3 of air / day / m^3 of bioreactor capacity

Expected Performance

Parameter	Blended Effluent 25% (50%) of HRAS
BOD_5 mg/L	80 (65)
TSS mg/L	80 (65)

Plant Footprint

IIWWTP: Approximately 20,000 m^2 is required to treat 50% of the ADWF
LGWWTP: Approximately 5,200 m^2 is required to treat 50% of the ADWF

Advantages

- Proven technology e.g. New York City
- Interim LWMP effluent goals met – BOD, TSS
- Re-use entire process – secondary build-out
- Odour control requirement limited
- Good quality biosolids
- Operating costs lower than CEP
- Smaller bioreactor volume than CAS

Disadvantages

- Slightly more sludge than CAS
- Removal efficiency lower than CAS by approximately 10%
- Significant plant footprint
- Additional staff required
- Different process than AIWWTP and LIWWTP
- Capital costs higher than CEP

Comments

- Good option for IIWWTP
- Probably poor choice for LGWWTP because of cramped site, unless some of the primary clarifiers are no longer used in process and HRAS can be built in existing clarifiers

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7.2.4 Roughing Trickling Filter (RTF)

Process Description

A summary description and process diagram of a roughing or ultra high rate trickling filter are provided in Figure 7.8. RTFs support hydraulic loading rates of 12-70 m^3/m^2-d. Ultra high-rate designs can support hydraulic loading rates of 47 – 188 m^3/m^2-d. The organic loading of BOD_5 is also high at 0.5-1.6 kg/m^3-d and 1.6-8 kg/m^3-d, respectively.

A common method of upgrading existing activated-sludge plants is to install a roughing filter ahead of the activated-sludge process. As part of the roughing filter activated sludge (RTF/AS) process, the roughing filter is typically 15 to 30% of the size that would be required if treatment had been accomplished through the use of the trickling filter process alone. The hydraulic retention time in the aeration basin is typically 35 to 50% of that required based on the use of the activated-sludge process alone.

Some TF plants have been built to operate with two or more TF units in series. These plants are called two-stage or multistage TF plants, if intervening clarification is included. Two filters directly coupled in series and operated at the same hydraulic rates typically perform as if they were one unit of the same diameter with the total depth of the two filters, especially if they have forced ventilation.

Under current practice, distinctions are made among TF applications based on the treatment provided rather than the hydraulic rate or organic loading of the application. This approach more accurately identifies the purpose of the TF operation. Hence, the general types of TFs are:

- Roughing filters that provide approximately 50 to 75% SBOD removal and 30 to 45% BOD_5 oxidation, followed by a second stage of treatment;
- Complete treatment filters that provide the required settled effluent BOD_5 and TSS;
- Combined BOD_5 removal and nitrogen removal filters that provide the required settled effluent quality for BOD_5, TSS, and ammonium-nitrogen; and
- Tertiary nitrifying filters that provide required effluent ammonium-nitrogen in a tertiary mode receiving a clarified secondary influent.

Adequately sized final settling tanks are required to achieve proper effluent levels of TSS and BOD_5. The application of modern and deeper clarifier designs with energy-dissipating, center-feed wells, baffled launders, and moderate overflow rates are the keys to good effluent quality.

Proven Technology

Technologies currently available can produce Advanced Wastewater Treatment (AWT) effluents of 10 mg/L BOD_5 and TSS or less and ammonium-nitrogen effluents of 1 mg/L or less. Trickling filters have historically been considered vulnerable to climatic changes because wastewater droplets must be exposed to large volumes of ambient-temperature air. However, proper engineering design can reduce temperature losses caused by wind and ventilation to less than 1.5°C. Improving dosing procedures and minimizing recirculation can also help to control temperature loss.

Temperature effects on nitrifying trickling filters are now considered to be less significant than those on activated sludge. Earlier observations of poor effluent quality in winter were caused by a combination of shallow filters with high surface area, low freeboard, and high recirculation ratios that caused excessive heat losses. Other conditions contributing to poor performance included poor clarifier designs and filter dosing procedures that caused excess solids accumulations.

Trickling filters are no longer viewed only as a process to produce secondary treatment effluent. The TF process now used for AWT produces low residual BOD_5, TSS, and ammonium-nitrogen. Replacing existing TFs is often more expensive than updating and expanding existing units using known process technology such as the addition of short-term aeration or the solids-contact process.

In applications where more stringent effluent quality standards have exceeded the capability of existing TF designs, expanding TF capabilities often meets the requirements. Based on recent experience, the full potential of the TF is only now being realized. The improved treatment capabilities of new and modified facilities, along with inherent ease of operation and low power use, have resulted in continued use of TFs.

Discharge Requirements/Effluent Quality

The RTF (excluding solids contact) in combination with secondary clarification will not exceed a 45/45 BOD/TSS effluent quality on a consistent basis. The quality can be further upgraded by the inclusion of solids contact tank between the trickling filter and the secondary clarifier.

Reliability

Successful conventional secondary and AWT applications are achievable with TFs but require a better understanding of TF operation and required appurtenances. If proper design procedures are used, TF performance equaling that of suspended-growth systems can be achieved:

- Trickling filters can produce effluent qualities of <10 mg/L BOD_5 and TSS;
- The effluent can be comparable to activated-sludge effluent;
- Trickling filters rapidly reduce soluble BOD_5 in applied wastewater;
- Temperature loss is less than 1.5°C in cold climates;
- Trickling filters are efficient nitrification units and effluents of <1.0 mg/L ammonium-nitrogen can be produced;
- Natural ventilation is inadequate for optimizing performance and power ventilation should be used;
- For rotating arm applicators, trickling filters should be dosed every 10 to 60 seconds, but routine flushing; at 10 to 30 minutes/dose is also needed to enhance performance; alternatively, solid set and pumped application has wider flushing capability;
- Recirculation is typically beneficial for optimum performance, especially if the hydraulic loading rate is low;
- Power consumption is typically 25% less than activated-sludge treatment;
- Trickling filter sloughing cycles are harmful to filter performance and can be avoided by daily flushing; and
- Less land area is required for TFs than for activated-sludge treatment.

Site Suitability

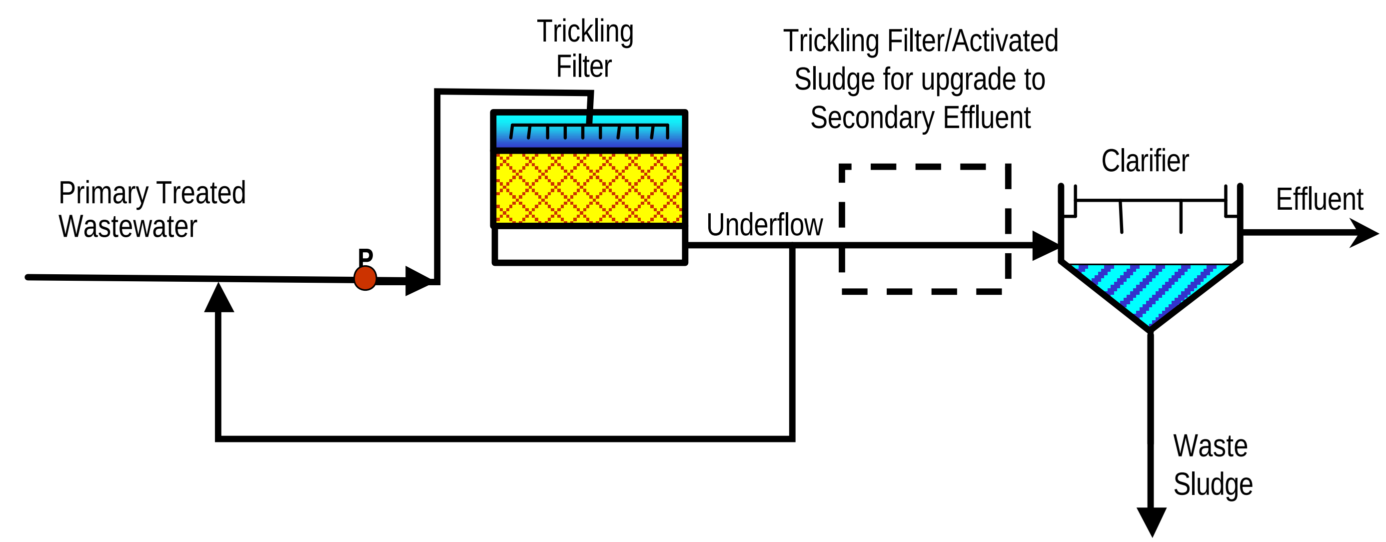
The Ultra High rate TF facility is suited to either the Iona or the Lions Gate sites. For Iona, the site is of sufficient size to easily accommodate the TF. For the either site the TF process could be added as a downstream process to the primary sedimentation tanks. At Lions Gate, a TF addition may need the incorporation of high towers since site space is limited.

FIGURE 7.8
ROUGHING OR ULTRA-HIGH TRICKLING FILTER

Process Description

(See also Trickling Filter - Standard Rate) Roughing filters are specially designed trickling filters, typically operated at high hydraulic loadings, necessitating the use of high recycle rates. They are used primarily to reduce organic loading on downstream processes and in seasonal nitrification applications. As with other biological processes, roughing-filter performance is temperature-sensitive. The higher hydraulic loadings of this kind of filter cause nearly continuous sloughing of the slime layer. If unsettled filter effluent is used for recycle, the sloughed biological solids in the recycle stream may contribute to organic removal within the filter as in a suspended-growth process.

Schematics



Design Criteria

Filter medium: Plastic / redwood / cedar
Hydraulic loading: 40 – 200 $m^3/m^2.d$
BOD loading: >3.0 $kg/m^3.d$
Depth: 0.9 – 6 m
Recirculation ratio: 1 – 4
Sloughing: continuous
Power: 8 – 16 kW/MLD

Expected Performance

Parameter	Blended Effluent 50% (Percent Removal)
BOD_5 mg/L	80 (40 – 70%)
TSS mg/L	70 (70 – 80%)

Plant Footprint

IIWWTP: Approximately 10,000m^2 is required to treat 50% of the ADWF
LGWWTP: Approximately 2,500 m^2 is required to treat 50% of the ADWF

Advantages

- Proven Technology
- Operating Costs Lower Than AS
- Simple operation
- Robust process resistant to toxic and hydraulic shocks
- Low energy requirement for aeration
- Biomass cannot be washed out by high peak flows
- Does not suffer from filamentous bacteria, sludge bulking, foaming
- Biomass has excellent settling qualities
- Smaller footprint than activated sludge
- Potential for partial treatment of full flow for MBAS removal

Disadvantages

- Possible psychoda (filterfly) nuisance
- Odour potential from trickling filter
- Can have High trickling filter profile (visual)
- Poorer quality effluent than TF/SC or activated sludge

Comments

- Good option for IIWWTP
- Probably poor choice for LGWWTP because of cramped site, unless some of the primary clarifiers are no longer used in process and HRAS can be built in existing clarifiers

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7.2.5 Biological Aerated Filter (BAF)

Process Description

The description below has been extracted from Water Environment Federation Manual of Practice No. 8, 4th Edition.

There are many innovations in the processing of wastewater using submerged fixed media. These systems can be sorted into two basic categories:

- Fixed film elements submerged in mixed liquor where there is a sludge return from the secondary clarifier. These elements may be suspended in the mixed liquor (for example, Captor®, KMT®, and Linpor®-C) or fixed (for example, Ringlace®, submerged RBCs, Bio 2, and Sludge). The fixed film may or may not play the dominant role in biological treatment, depending on the design.
-
- Fixed film elements and attached biomass are the primary mechanisms of the treatment process. Liquid may be recycled, but clarified sludge is not. These processes may use floating (Biostyr®), subsided bed (for example, BioCarbone®, Biofor®), or fluidized-bed (for example, Oxitron®, Biolift®) media.

These processes have been used for BOD_5 removal, nitrification, and denitrification of municipal and industrial wastewater. A general objective of these processes is to complete the biological treatment in less space, and these designs may or may not be less costly. Future improvements in these processes are to be expected as experience is gained. This section will review only the fixed film processes that do not recycle sludge. The first group of processes, which are a combination of the fixed and suspended-growth biology, are discussed in Section 4.4 of Appendix #4.

There are several developed processes and many technology programs underway throughout the world that relate to enhancing the performance of submerged fixed film reactors. Of the processes only a few are currently in commercial use. A detailed description of the various BAF processes can be found in Section 4.1.5 of Appendix 4.

Because of readily available information and proven track record the Biofor® process has been selected as an example of the type of process which could be utilized. The process ultimately chosen should be selected based on a full evaluation of the available candidates at the time.

Proven Technology

The family of biological aerated filter processes has been used for more than 10 years. There are in excess of 200 installations around the world with at least 5 in Canada. Of the plants treating municipal wastewater the capacities vary up to 410 ML/d PWWF. Reference sites are as follows:

Biofor® – 68 Plants worldwide with 12 in range 100 to 1,700 ML/d of which 2 are in Canada (City of Quebec and Thunder Bay). One of the 12 is a BOD removal plant with the remainder being for tertiary nitrification.

Discharge Requirements/Effluent Quality

The effluent quality should comfortably meet the required 45/45 secondary standard. Should ammonia conversion to nitrate be required this could be achieved by adding on additional units. De-nitrification, likewise, could be achieved.

Reliability

Problems, which have occurred during early full-scale implementation, have been addressed progressively over time. Careful selection of an appropriate proprietary configuration should provide the necessary level of assurance. Changes in influent sewage characteristics should be accommodated by the adaptation of the biomass over a short time. Daily fluctuations in flow and load would be accommodated by varying the number of units online at any time. The plant is subject to the disruption of the control systems.

Site Suitability

These plants have a small footprint and are therefore particularly suited to the Lions Gate WWTP site.

BIOLOGICAL AERATED FILTER (BAF)

FIGURE 7.9
BIOLOGICAL AERATED FILTER (BAF)

Process Description

Primary effluent is pumped upwards or downwards through a bioreactor containing fixed media on the surface of which biomass grows. Essentially the proprietary bioreactor is a submerged aerated fixed film reactor. Air is injected in the form of fine bubbles, 1-2 mm in diameter near the base of the media in co-current flow with the primary effluent inlet stream. The biomass utilizes the organics in the wastewater as food and converts them to CO_2, water and additional biomass. The media is approximately 3 to 4.0 metres deep, has a high specific surface area, high porosity and is manufactured from materials which are resistant to attrition (e.g. Biofor® media consists of an expanded clay material). Periodically the bio-filters are backwashed and simultaneously agitated by air scour to wash biosolids from the media. Filter effluent is stored to provide backwash water. The backwash cycle can be controlled by a timed cycle and or head loss measurements. Multiple cells are utilized and can be cycled in and out of service to ensure generation at optimum flow rates for biological growth through a range of plant flows and load conditions.

Design Criteria

Total organic loading: 4~6 kg CBOD/$m^3 \cdot d$
Hydraulic loading: 132 $m^3/m^2 \cdot d$ (average flow), 480 $m^3/m^2 \cdot d$ (peak flow)
Media bed depth: 4 m
Overlying liquid depth: 1.5 m
Hydraulic retention time: 0.5 - 2.0 hours
Media bed backwash: 0.2~0.6 hr / 1~ 2 days
Backwash storage: 10% area of BAF reactor
Backwash equalization: 20% area of BAF reactor
Air requirement: 1.2 kg O_2 / kg BOD_5 removed

Expected Performance

Parameter	Effluent 50% (Percent Removal)
BOD_5 mg/L	60 (> 90%)
TSS mg/L	55 (> 90%)

Plant Footprint

IIWWTP: Approximately 12,000 m^2 is required to treat 50% of the ADWF
LGWWTP: Approximately 2,800 m^2 is required to treat 50% of the ADWF

Schematics

Figure 7.9.1
The BioCarbon downflow biological process

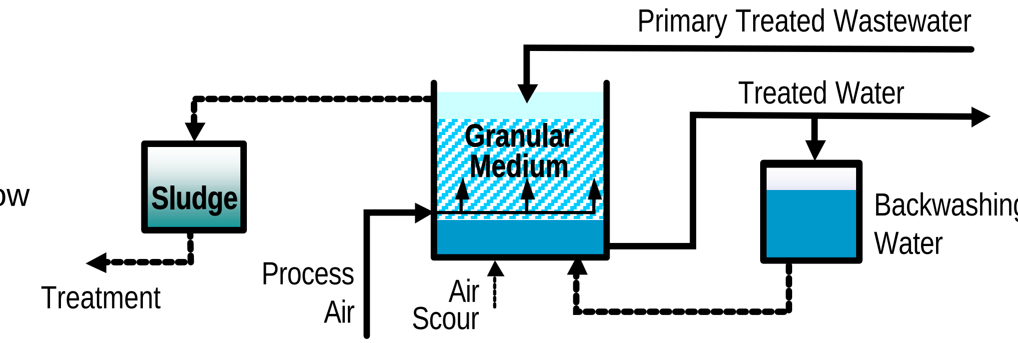


Figure 7.9.2
The Biofor® upflow biological reactor

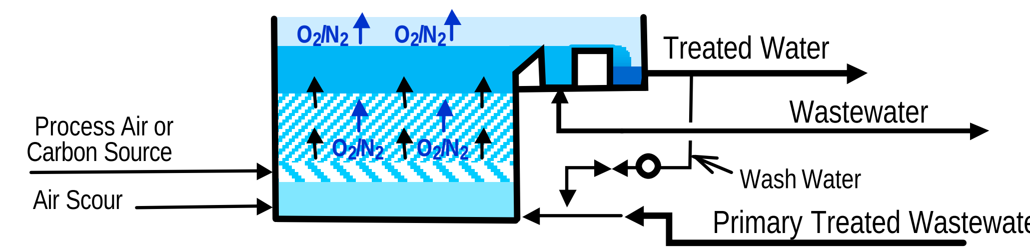
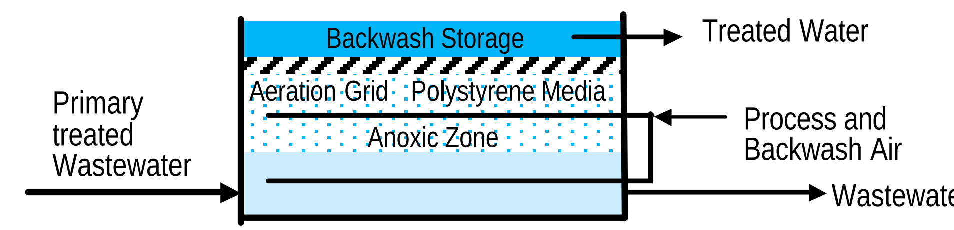


Figure 7.9.3
The BioStyr arrangement for nitrification-denitrification



Advantages

- No Final Clarifiers Required
- Smaller Footprint (~1/3) of CAS
- Excellent BOD and TSS Removal
- Easy to Upgrade for Nitrification / Denitrification (Add on BAF Cells)
- Odour Minimized
- Automated Operation
- Biosolids Amenable to Anaerobic Sludge Stabilization
- Sludge Quality about the same as CAS
- Can Easily be applied after CEP - Reduce Overall Capital Cost
- LWMP Effluent Goals Met – BOD, TSS
- Toxicity Reduced to LC_{50} of 100% Effluent

Disadvantages

- Limited Application in Canada
- Proprietary Technology
- High Energy Requirement (~40% more than CAS) - Aeration Pumps
- Effluent Storage and Pumping Required
- Backwash Cycle Load to the Primary Clarifiers
- Capital Cost Higher than CAS and TFSC ≈ 25%
- Need to Design for Peak Day Flows
- Sensitive to High Influent Loads
- High Backwash Flow (up to 20% of Influent Flow)
- Different Process than Annacis / Lulu

Comments

- Excellent candidate for application at Lions Gate or other sites with limited area. Probably not an economically competitive alternative for Iona Island.
- Currently being designed for use in conjunction with CEP at West Windsor Ontario

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7.3 CEP WITH PARTIAL BIOLOGICAL TREATMENT

7.3.1 Process Description

The combination of chemical enhanced primary (CEP and its modifications, Section 7.1) and partial biological treatment (Section 7.2) can be considered as an interim upgrade option to achieve BOD_5 and TSS, as well as effluent toxicity reduction. A process schematic is illustrated in Figure 7.10, in which those CEP and biological treatment options can be fit in the designated locations in the process. This option consists of treating 100% of the flow with CEP followed by biological process (such as RTF) to treat 50% of the CEP effluent (Figure 7.10a).

Detailed process descriptions of CEP and biological treatments were discussed in Section 7.1 and 7.2, respectively. The CEP process will improve the TSS and BOD_5 removal efficiency in the primary treatment. A portion of the primary effluent can be treated in the biological process to achieve additional BOD_5 removal, and sequentially TSS removal in the secondary clarifiers. Operational conditions of CEP and biological treatment should be designed and controlled to accommodate the system requirements, including chemical dosage in CEP, F/M ratio, SRT, and aeration in the biological process. CEP can be operated on a continuous or intermittent basis (e.g. during peak loads and dry weather), however, the biological system should preferably be operated continuously to maintain the stability of the system. CEP can also be operated to accommodate the biological system needs (e.g. organic loads) and effluent quality requirements (i.e. TSS and BOD_5 concentrations).

Sludge production will increase with CEP and partial biological treatment, compared with conventional primary settling (current condition), or CEP and biological treatment alone. This is discussed in more detail in Appendix 7. Different types of sludge will be generated, including primary sludge, chemical sludge, biological sludge, or their combinations. As a result, solids handling requirements and disposal options will be more complex. Combined or separated sludge treatments need to be considered and carefully planned in accordance with this interim upgrade option. Odour is generally not a problem at plants utilizing CEP and aerobic biological processes.

7.3.2 Proven Technology

CEP and biological treatment alone have been widely used in municipal wastewater treatment, but they are not commonly operated in combination due to their different natures (chemical vs. biological) and the complexity of sludge handling. CEP is usually operated as the prime process, or as an interim option before the full secondary upgrade (i.e. biological treatment) is in place. Intermittent CEP may be a good option to operate in conjunction with biological treatment (prime process) as mitigation when system upset or shock loads are experienced.

7.3.3 Discharge Requirements/Effluent Quality

With proper chemical dosage and a biological system the interim effluent quality (130 mg/L of BOD and 100 mg/L of TSS) can be met with CEP and partial biological treatment. This combination is expected to deliver 70 mg/L of BOD_5 and 70 mg/L of TSS in the effluent, respectively.

7.3.4 Reliability

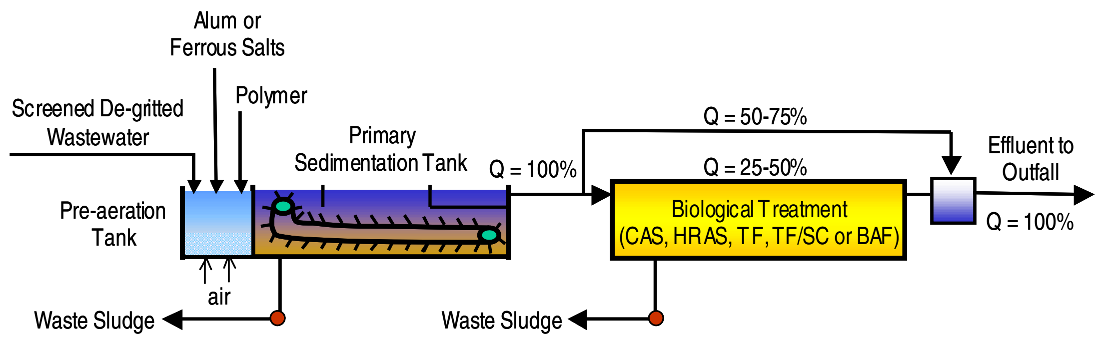
The CEP process can be easily started up, but the biological treatment needs several days or a week to recover following system upsets. The operational condition of CEP can be adjusted to mitigate the effluent quality should deterioration occur.

7.3.5 Site Suitability

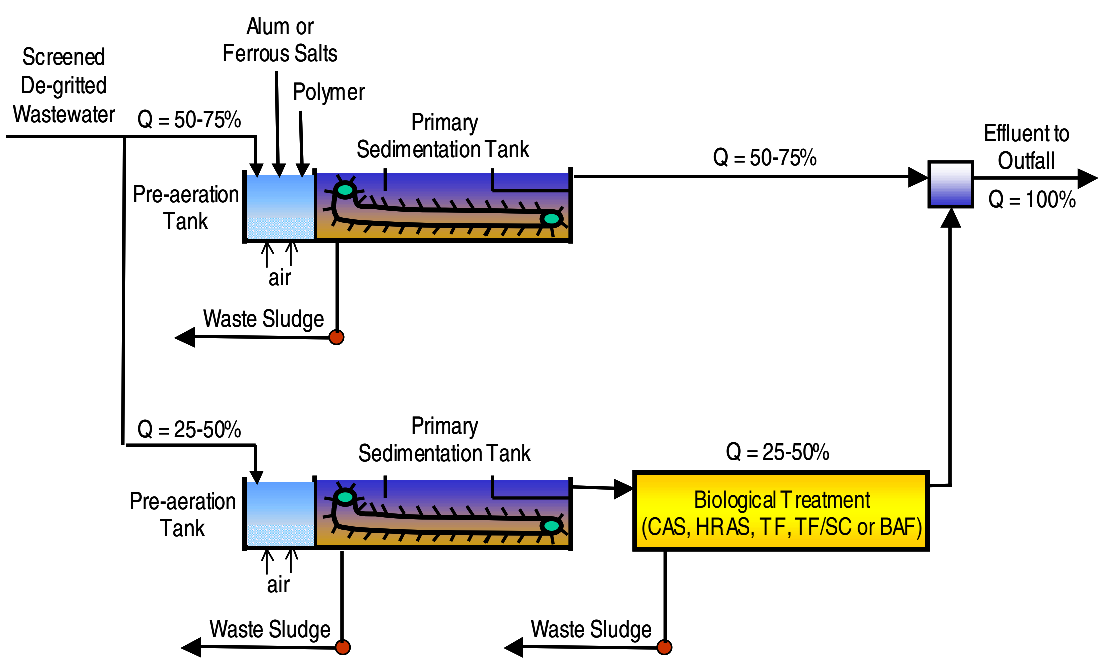
Significant footprint expansions are expected at both treatment plants, primarily due to the space requirements of the biological treatment units and solids handling facilities (i.e. solids thickening and digestion). This interim upgrade may be a good option for IIWWTP, provided that the biological treatment can be extended for the future secondary build-out. The capacity of solids handling should be upgraded accordingly to suit the process needs. Limited space at LGWWTP may mean that opportunities for the implementation of this process will be limited.

FIGURE 7.10
CEP WITH PARTIAL BIOLOGICAL TREATMENT SCHEMATICS - GENERAL

(a)



(b)



7.4 DISSOLVED AIR FLOTATION (DAF)

7.4.1 Process Description

Dissolved air flotation (DAF) removes solids from liquids by floating and skimming off solids attached to air bubbles. In the DAF process compressed air is introduced at a controlled rate into pressure tanks and mixed with a recycle stream (e.g. primary effluent). It is then discharged into the DAF tanks where micro-bubbles are formed as the pressure is released. The rising bubbles adhere to solids and greases and form a floating blanket at the surface. This is conveyed towards one side of the DAF tank where the "float" or sludge blanket is skimmed off into a hopper. The formation of a thick, floating sludge blanket can be aided by addition of alum, ferric chloride and/or polymers. A process schematic and technical information are summarized in Figure 7.11.

DAF is capable of achieving high solids removal (up to 85% TSS), which can be added to treat part of the degritted flow to enhance the overall effluent quality. Typical solids and hydraulic loading rates are 30 - 60 $kg/m^2/d$ and 190 - 280 $m^3/m^2/d$, respectively. Higher loading rates can be operated if chemical aids and polymer are used. In comparison with conventional primary settling, more sludge production is expected (e.g. increase by 25 ~ 50%) due to higher removal efficiency and chemical aids additions. Odour is generally not a concern in DAF operation, however enclosure and off-gas treatment can be implemented for odour mitigation.

7.4.2 Proven Technology

DAF is commonly used to achieve high rate solids removal and sludge thickening worldwide. With proper loading design and operational control, high efficiency of solids capture can usually be assured. DAF process is a proven technology and common unit operation in North America water and wastewater treatment plants. DAF has been operated in AIWWTP and LGWWTP for sludge thickening to achieve more than 90% of solids capture. However, it is not normal to use DAF as a method for sewage solids reduction due to the relatively low solids concentration.

7.4.3 Discharge Requirement/Effluent Quality

TSS removal using DAF is efficient. Minimum removal levels are normally 85 ~ 90%. However, the capability of BOD_5 removal is limited to the organic contents associated with solids only. Using DAF to treat part of the degritted flow (e.g. 50%), the overall effluent quality is expected to be 90 mg/L for BOD_5 and 70 mg/L for TSS. As with all non-biological processes, the improvement of effluent toxicity reduction may be marginal because of limited soluble substance removal in DAF. In general, DAF will improve the removal of suspended solids and MBAS, but with limited BOD_5 removal enhancement. The ability to achieve adequate removal of MBAS has yet to be demonstrated.

DISSOLVED AIR FLOTATION (DAF)

Process Description

Primary effluent from the existing primary settling tanks will be discharged to a dissolved air flotation (DAF) tank. High pressure water with dissolved air is introduced at the bottom of the tank. Suspended solids will attach to the rising air bubbles, float to the surface and be removed by skimmers or scrapers at the top of the tank. The treated effluent will be collected in the bottom of the tank. Polymers can be added to aid flocculation of solid particles.

DAF is capable of achieving high TSS removal (minimum 85 ~ 90%), which can be added to treat part of the degritted flow to enhance the overall effluent quality. In comparison with conventional primary settling, more sludge production is expected (e.g. increase by 25 ~ 50%) due to higher removal efficiency and chemical aids additions. Odour is generally not a concern in DAF operation, however enclosure and off-gas treatment can be implemented for odour mitigation.

Design Criteria

Tank depth: 3 ~ 4 m
Hydraulic loading rate: 190 ~ 280 m^3/m^2/d
Solids loading rate: 30 ~ 60 kg/m^2/d
Polymer addition: optional

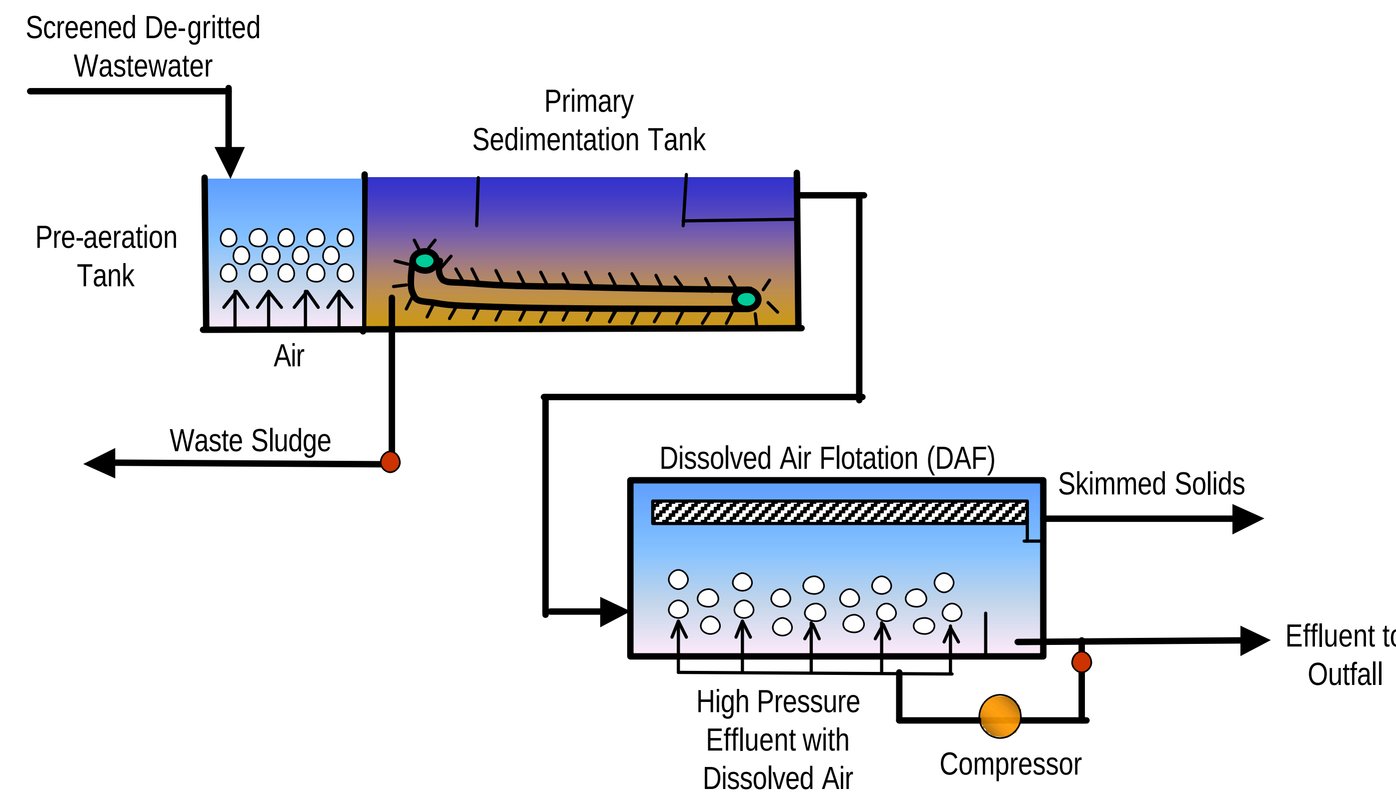
Expected Performance

Parameter	Blended DAF Treated Effluent (50%)
BOD_5 mg/L	90
TSS mg/L	55

Plant Footprint

IIWWTP: Approximately 1,800 m^2 is required to treat 50% of the flow
LGWWTP: Approximately 450 m^2 is required to treat 50% of the flow

Schematics



Advantages

- Proven technology
- High solids removal efficiency
- Interim LWMP goals met – BOD, TSS
- Toxicity reduced to LC_{50} of 100% effluent
- Small footprint
- Potential to remove MBAS

Disadvantages

- No/low BOD removal
- High energy consumption
- No advantage for application at Iona Island
- Different process than Annacis Island and Lulu Island

Comments

- Could be a candidate for Lions Gate because of small footprint, better MBAS removal than CEP, and aesthetics of bioreactor.
- Not a candidate for Iona.

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7.4.4 Reliability

The operation and performance of DAF are generally considered reliable in achieving the desired removal efficiency. DAF can be brought online within fairly short period of time (e.g. hours). Additional chemical aids can be applied during the transition to control the effluent quality.

7.4.5 Site Suitability

DAF has a small footprint and could be a good candidate for IIWWTP as well as LGWWTP. With DAF to treat part of the degritted flow (e.g. 50%), approximately 1,800 m^2 and 450 m^2 of extra footprint are required for IIWWTP and LGWWTP, respectively. Digester capacity needs to be expanded in order to process the additional solids production. DAF requires a significant amount of capital investment in mechanical equipment including pumps, compressors, saturation tanks, chemical feed systems and controls. As well as being capital cost intensive, skilled operator(s) are required and high-energy consumption is to be expected with the DAF process. However, these DAF units could be used for sludge thickening in the future secondary build-out.

7.5 PRIMARY TREATMENT WITH ADD-ON CHEMICAL TREATMENT

7.5.1 Chlorination and Dechlorination

Process Description

Chlorination, in conjunction with dechlorination, has been widely used in wastewater treatment for various purposes which include odour control, sludge conditioning, disinfection, BOD reduction, and ammonia oxidation. The process utilizes the high oxidizing power of chlorine or its compounds to react with organic or odorous substances including microorganisms. Dechlorination follows chlorination to remove the chlorine residual in the effluent in order to preclude any environmental impact (e.g. oxygen demand). The objective of this treatment is to eliminate microscopic (activated sludge) organisms that have grown in the sewers or at the plant and which would have the potential to reduce the dissolved oxygen content of the effluent. In addition, chlorination will result in some BOD reduction. A process schematic and technical information are summarized in Figure 7.12.

Gaseous chlorine (or chlorine compounds) is added as an oxidizing agent to the primary effluent at approximately 5 to 20 mg/L in a contactor where the effluent is retained prior to discharge for 1 - 1.5 hours. A high percentage of pathogenic bacteria will be removed. The number of faecal coliforms will typically be reduced from the 10^7/100 mL range to around 10^5/100 mL. Any residual chlorine remaining after the contactor will be dechlorinated by the addition of sulphur dioxide, as reducing agent.

Chemical treatment using chlorine may assist in improving LC_{50} test results by reducing the oxygen demand exerted by microorganisms. The capital investment includes chemical dosing equipment, chemical storage facilities, contact tanks, and gaseous monitoring instrumentation (if gaseous chlorine is used). Operating costs include only chemical supply costs. Pre-chlorination can be carried out at the pre-aeration tanks to achieve odour control and microorganism growth inhibition. Dechlorination can be provided in the effluent channels. Due to the nature of the chemical reactions, there will be no additional TSS removal and sludge production.

Proper measures must be taken to ensure the safety of operators and general public. Alternate sources of chlorine such as sodium or calcium hypochlorite solutions can be used to allay concerns regarding the use of gaseous chlorine.

FIGURE 7.12
CHLORINATION AND DECHLORINATION

Process Description

Gaseous chlorine is added to the primary effluent at approximately 5 to 20 mg/L prior to discharge. A contact time of 1 –1.5 hours is provided in the chlorine contactor. Any residual chlorine remaining after the contactor will be dechlorinated by the addition of sulphur dioxide so that chlorinated organics do not continue to form in the outfall en-route to the sea. The objective of this treatment is to kill off any microscopic (activated sludge) type organisms that have grown in the sewers or at the plant. A certain level of BOD reduction will be achieved with chlorination. The process will also kill a high percentage of pathogenic bacteria in the primary effluent reducing faecal coliforms from the 10^7/100 mL range to around 10^5/100 mL. Essentially this is a chemical treatment assisting in enabling the effluent to always pass the toxicity tests.

Design Criteria

Chlorine contact time: 1 –1.5 hours
Chlorine dosage: 5 to 20 mg/L
Gas chlorination
Sulphur dioxide addition: 1:1 ratio with residual chlorine

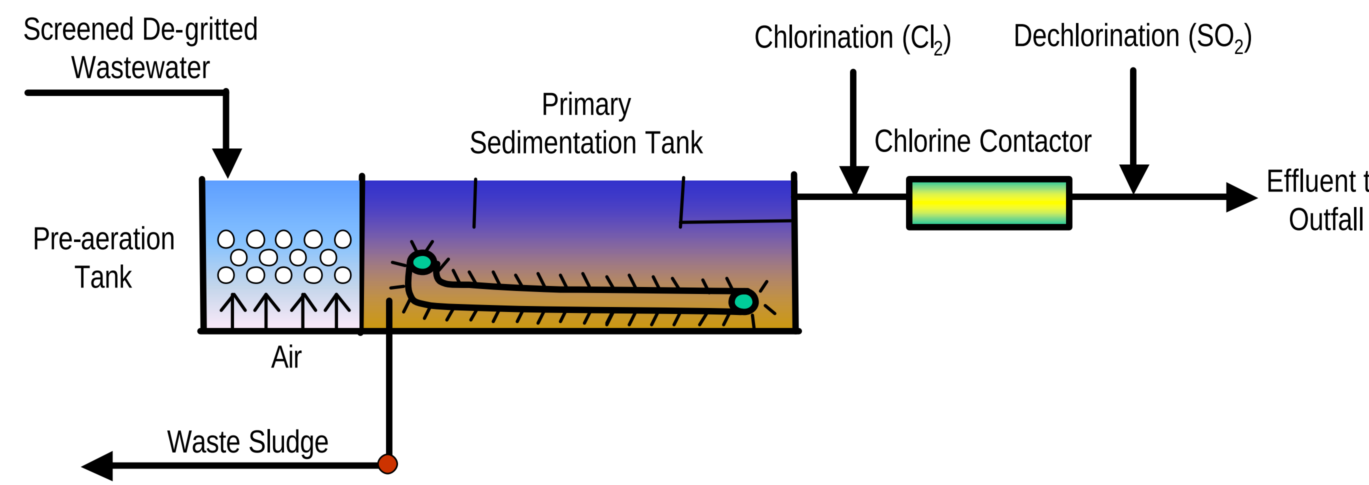
Expected Performance

Parameter	Blended Final Effluent
BOD_5 mg/L	75
TSS mg/L	90

Plant Footprint

Small footprint is required for chemical dosing devices and chemical storage.

Schematics



Advantages

- Low capital cost
- Low operating cost
- Improved microbiological effluent quality
- Easy to operate
- No additional sludge generation
- Iona samples will pass toxicity tests
- Odour control benefit if applied to pre-aeration

Disadvantages

- Opposition from environmental groups to chlorine addition
- Operational concerns of handling gaseous chlorine chemicals
- Regulatory approval will be difficult
- No additional TSS removal
- No improvement in LC_{50} for Lions Gate
- Public concern over transport of gaseous chlorine

Comments

- This is an option for Iona Island to improve the LC_{50} test results
- No impact on MBAS toxicity at Lions Gate

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Proven Technology

Chlorination and dechlorination are among most commonly applied and cost effective disinfection treatments. The long history of such application has demonstrated the efficacy of this technology for microorganism control as well as for many other purposes. Emerging concerns about potential environmental impacts of DBP have drawn attention to the importance of correctly designing the chlorine injection point and need for dechlorination. With proper process control, high efficiency of microorganism removal can be achieved and the formation of chlorine residuals can be minimized.

Discharge Requirement/Effluent Quality

Chlorination and dechlorination will not result in any additional TSS removal. Effluent toxicity reduction may be achieved as a result of microorganism inhibition and possible additional BOD_5 removal (organic oxidation) at IIWWTP.

Reliability

Chlorination and dechlorination are considered to be a reliable process. It can be brought online within minutes if the dosing devices are functioning. They are also easy to operate and maintain. Dosage rates can be adjusted to accommodate flow and load variations. Online monitoring of chlorine and sulphur dioxide concentrations is commercially available for dynamic process control and effluent quality assurance (e.g. chlorine residual).

Site Suitability

A small footprint is required for the chemical dosing devices and for chemical storage facilities. The system could easily be installed at both treatment plant sites. Capital and operating costs would be low compared to chemically enhanced primary and biological treatment. Currently, chlorination and dechlorination is seasonally implemented at LGWWTP.

7.5.2 Ozonation

Process Description

Ozonation is an emerging process that has been used increasingly in water and wastewater treatment. Significant improvements in ozone generation techniques in the past decades have made ozonation more applicable at large-scale installations. Uses include odour control, colour removal, disinfection, and advanced organic oxidation. Ozone is a strong oxidizer that can be added to wastewater in the gaseous form to react with organic substrates. The key to the use of ozone is the oxidizing potential of free hydroxyl radicals (·OH) that are formed when ozone decomposes in a series of chain reactions in water. Hydroxyl radicals are able to convert organic substrate to carbon dioxide or water and mineral salts. Ozone is also an effective disinfectant, which can efficiently eliminate live microorganisms.

Ozone can be applied to the primary effluent at existing plants to remove organic substrate (BOD_5) and microorganisms. Process schematic and technical information are summarized in Figure 7.13. An ozone contact basin would be added after the primary sedimentation process. Ozone can be generated using an ozone generator in which a high voltage alternating current (typical 6 ~ 20 KV) is applied across a dielectric discharge gap that contains injected dry air or oxygen gas. Ozone is injected into the contact basin through a fine bubble diffuser. Primary effluent will be mixed with ozone in the contact basin where further BOD removal and microorganism destruction take place. The off-gases from the contact basin will be recycled to the contact basins or destroyed by reducing agents.

Apart from stand-alone application, ozone can be used in conjunction with high oxidizing potential chemicals such as hydrogen peroxide (H_2O_2) in an advanced oxidation process (AOP). In the process, ozone is activated by hydrogen peroxide to generate hydroxyl radicals:

$$2\ O_3 + H_2O_2 \rightarrow 2\ \cdot OH + 3\ O_2$$

The highly reactive hydroxyl radicals result in the breakdown of organic pollutants to carbon dioxide and water. The removal efficiency of BOD is higher using AOP compared to the use of ozone alone.

Ozone is unstable in nature and needs to be generated on-site. As a result, safety problems normally associated with the shipping and handling of chemicals are eliminated. Unlike chlorine, when used as a disinfectant, ozone does not produce chlorinated organics or disinfectant by-products (DBP). The major drawback to ozonation is the high capital cost of the ozone generation equipment and power consumption in operation. Thus, ozonation is not economical for wastewater with high levels of solids and organics.

Process Description

Ozone is a strong oxidizer that can be added to wastewater in gaseous form to react with organic substrates. The key to the removal capacity of ozone is the oxidizing potential of free hydroxyl radicals (·OH) that are formed when ozone decomposes in a series of chain reactions in water. Hydroxyl radicals are able to convert organic substrate to carbon dioxide or water and mineral salts. Ozone is also a disinfectant, which can efficiently destruct live microorganisms. An ozone contact basin, where ozone is injected, will be added after the primary sedimentation process. Primary effluent will be mixed with ozone in the contact basin where further BOD removal and microorganism destruction take place. The off-gases from the contact basin will be recycled to the contact basins or destroyed by reducing agents.

Design Criteria

- Contact time: 10~30 minutes
- Ozone dosage: 5 ~ 25 mg/L (pilot-test required to determine optimum dosage)

Expected Performance

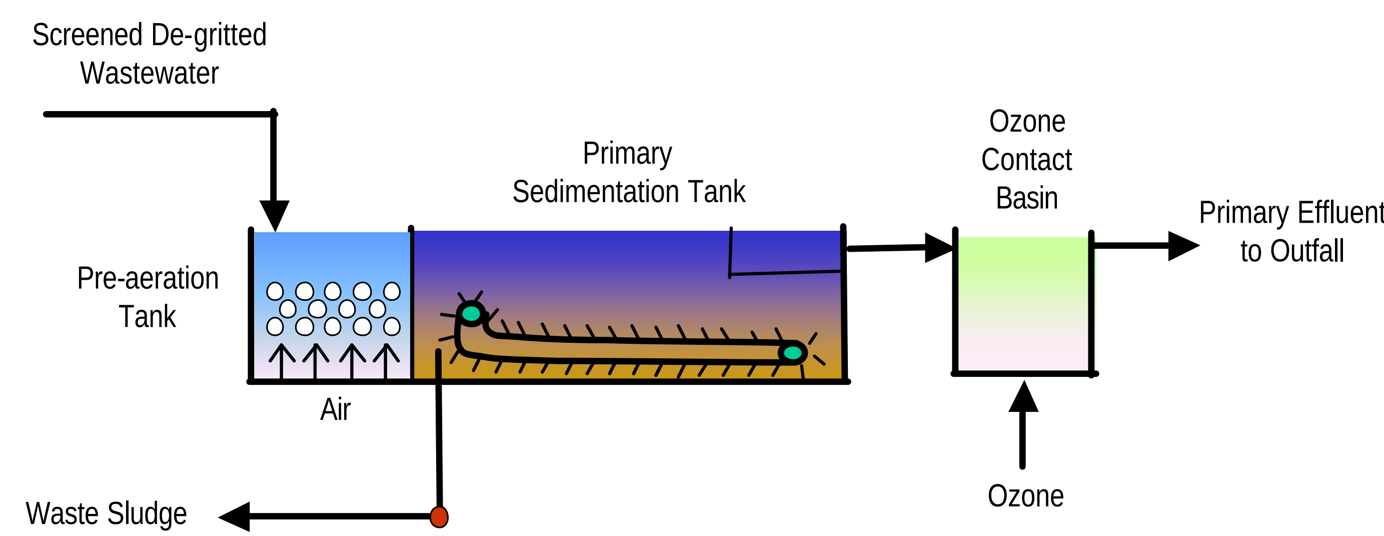
The performance of ozone is subject to chemical dosage, wastewater composition, effective contact time, pH, and temperature.

Parameter	Blended Primary + ozone Treated Effluent
BOD_5 mg/L	75
TSS mg/L	90

Plant Footprint

Small footprint is required for gas storage, ozone generation, dosing devices and off-gas control/monitoring.

Schematics



Advantages

- Small footprint requirement
- Reduce BOD and effluent toxicity
- Provide odour control, colour removal, and disinfection
- No disinfectant by-product in comparison to chlorine

Disadvantages

- Limited application in North America
- High capital cost
- High energy consumption
- Limited effluent quality improvement

Comments

- Can be applied at both LGWWTP and IIWWTP but not a competitive option due to limited effluent quality improvement, high capital costs of ozone generator, and high energy consumption in operation.

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Proven Technology

Ozone has had limited application in North America although this technology has been widely used in Europe for decades, particularly for drinking water treatment. Ozone has increasingly been considered as an alternative to conventional chlorination, mainly due to concerns regarding the formation of disinfectant by-product (DBP). GVRD has commenced the construction of a drinking water disinfection ozonation facility in Coquitlam, BC.

Discharge Requirements/Effluent Quality

Ozonation does not result in any additional TSS removal but there is some reduction of BOD_5 and, to a certain degree, effluent toxicity can be reduced.

Reliability

Due to the instability of ozone gas, the reliability of ozonation is highly dependent on the on-site ozone generating capacity. Typical commercial ozonators can be operated to reach steady ozone production within an hour, and the ozone dosing rate can be varied within the design range by adjusting the energy input.

Site Suitability

Ozonation can be applied at both LGWWTP and IIWWTP with a small footprint requirement for gas storage, ozone generation, dosing devices and off-gas control and monitoring. However, due to the limited effluent quality improvement, high capital costs of the ozone generator, and high operating energy consumption, ozonation does not appear to be a competitive option.

7.5.3 Hydrogen Peroxide

Process Description

Hydrogen peroxide is a strong oxidizer and has been used to remove colour (bleaching) and organic substrates in wastewater treatment. Removal of BOD by hydrogen peroxide is achieved mainly through direct chemical oxidation and potentially by physical flotation. Hydrogen peroxide oxidizes organic and inorganic substrates, completely or partially to carbon dioxide and water. The extent of the oxidation depends on the dosage rate. If more resistant (recalcitrant) substances need to be broken down, catalysts such as iron salt will be required. In addition, when hydrogen peroxide decomposes to oxygen and water (as illustrated by the following reaction), the gaseous oxygen (in the form of rising bubbles) may cause the flotation of fats, oils and greases which may in turn increase BOD removal in a similar manner to dissolved air flotation (DAF).

$$2\ H_2O_2 \rightarrow 2\ H_2O + O_2$$

Hydrogen peroxide addition is an add-on chemical process to be operated in conjunction with the existing primary treatment system or one of its modifications. A process schematic and technical information are summarized in Figure 7.14. The theoretical hydrogen peroxide requirement is about 2.1 kg (as 100% solution) per kg-BOD to be oxidized. Hydrogen peroxide can be injected into the wastewater stream either in the pre-aeration tanks or the primary effluent channel. No additional tankage is required. In addition to organic substrate removal, hydrogen peroxide can also result in odour control, microorganism inhabitation, and disinfection. However, the efficiency of hydrogen peroxide systems for different objectives is highly subject to the operating conditions, which include pH, temperature, and contact time. Optimum conditions should be determined by pilot tests, and adjusted accordingly based on flow and load.

Hydrogen peroxide can be used alone, or as a catalyst in conjunction with ozone in the advanced oxidation process (AOP). Hydrogen peroxide is readily available in bulk quantities in concentrations of 35% or 90% (industrial grade) by weight, but requires special handling precautions. Gaseous release is not a concern when liquid hydrogen peroxide is used.

Proven Technology

Hydrogen peroxide became commercially available in the 1800's and has been used in municipal wastewater applications for several decades. The most common uses are for odour control, colour removal, and auxiliary treatment of BOD removal. Hydrogen peroxide has also been successfully applied to supply dissolved oxygen and inhibit microorganism growth.

Discharge Requirements/Effluent Quality

Hydrogen peroxide may improve BOD removal and effluent toxicity reduction, but provides no additional TSS removal.

Reliability

Hydrogen peroxide is a useful oxidizing agent. With a well designed system it can quickly be brought online. It is considered a reliable process under a variety of flow and load conditions.

Site Suitability

Hydrogen peroxide can be introduced at both LGWWTP and IIWWTP as an add-on process for BOD and effluent toxicity reduction. Only a small footprint is needed for chemical solution storage and dosing equipment. The process does not require additional tankage. For these reasons, it could be applied at the Lions Gate site where additional space is limited.

FIGURE 7.14
HYDROGEN PEROXIDE

Process Description

Hydrogen peroxide addition is an add-on chemical process to be operated in conjunction with the existing primary treatment or its modification. Hydrogen peroxide has been used in municipal wastewater applications for several decades for odour control, colour removal, auxiliary treatment of BOD removal, supplying dissolved oxygen and microorganism growth inhabitation. Hydrogen peroxide is a strong oxidizer that oxidizes organic and inorganic substrates in wastewater to carbon dioxide and water. Oxygen (in the form of rising bubbles) produced during oxidation may cause the flotation of fats, oils and greases and increase BOD removal, similar to the dissolved air flotation (DAF) principle. Hydrogen peroxide can be added either in the pre-aeration tanks or the primary effluent channel. No additional tankage is required. Hydrogen peroxide may provide BOD removal and effluent toxicity reduction, but not additional TSS removal.

Design Criteria

- Pilot-test required to determine optimum dosage

Expected Performance

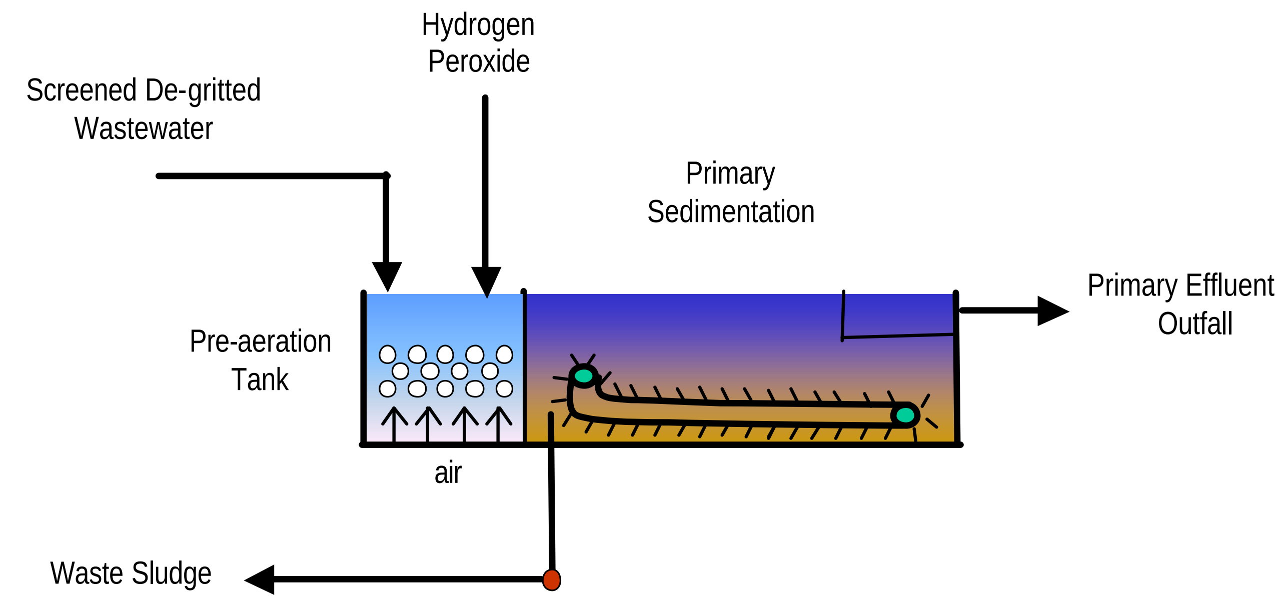
The performance of hydrogen peroxide is subject to chemical dosage, wastewater composition, effective contact time, pH, and temperature.

Parameter	Blended Primary + H_2O_2 Treated Effluent
BOD_5 mg/L	85
TSS mg/L	90

Plant Footprint

Small footprint is required for chemical solution storage and chemical dosing equipment.

Schematics



Advantages

- Improve BOD removal
- Reduce effluent toxicity
- No additional tankage required
- Proven technology
- Odour control
- Disinfection

Disadvantages

- No additional TSS removal
- Chemical storage on-site
- Special handling required

Comments

- Could be applied at the space-constraint Lions Gate site because only a small footprint is needed for the chemical solution storage and chemical dosing equipment

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8 FIRST LEVEL OF SCREENING AND RANKING

8.1 DESCRIPTION OF SCREENING PROCEDURE

The ability to make an informed choice between available processes depends on the availability of objective and comparable information. This need was addressed by preparing process descriptions in a format, which set out the key requirements as listed under Pass or Fail Evaluation below. These descriptions are included in Section 7 of the Appendix 3 for interim upgrades and in Appendix 4 for build-out to secondary options. A broad range of commercially exploited processes was included in the evaluation.

The initial screening was based on a short list of criteria identifying the key requirements of the processes for each plant. The initial level of screening was carried out in two phases, as described below.

8.1.1 Pass or Fail Evaluation

- Proven Technology - State of development. Is the process well established with several examples at an appropriate scale or is it newly commercialized/still in the research phase?
- Discharge Requirements - Ability to meet the discharge requirements (including achieving improved LC50 test results).
- Reliability of process - Ability to recover from an upset.
- Site Suitability - based on site constraints (Lions Gate only).

Each process was evaluated against each of the criteria on a simple pass or fail test. Only those processes, which passed all criteria, were considered for further evaluation unless there were special factors, which warranted further evaluation.

8.1.2 Preliminary Ranking and Elimination of Less Suitable Processes

Further evaluation was carried out against the following criteria:

- Capital Cost (Present Value)
- Operating Cost (Present Value)
- Reliability - with respect to meeting standards
- Integration - Ease of integration into present and future processes
- Flexibility - in accommodating changes in discharge standards
- Environmental (impact) – Energy efficiency, resource consumption, sludge quality/quantity, air quality, residuals production
- Social – Safety, visual impact, and odour.

Each process was evaluated against each of the criteria based on a 1 to 5 scale with 5 being the best. Each criterion had equal weighting in summing the ratings. The processes with the highest total ratings were considered further in a more detailed evaluation and ranking process.

8.1.3 Application of the Screening Procedure

The process descriptions were written using the four Pass / Fail criteria as headings. These descriptions were consulted during the procedure to establish whether a process would pass or fail. Where the process failed it was examined to see if there were special circumstances, which could warrant it being further considered, notwithstanding the failure. Reasons for further consideration could be that the process is not yet commercialized but is showing promise. The results of this Pass/Fail exercise were presented to the experts, identified below, for confirmation. Following this a Delphi evaluation was applied to the remaining processes.

The evaluation procedure used has been referred to as a "Delphi" exercise. The objective of the Delphi method is the production of suitable information for decision making. The Delphi method is based on a structured process for collecting and distilling knowledge from a group of expert by means of questionnaires interspersed with controlled opinion feedback. The technique allows experts to deal systematically with a complex number of options. The essence of the technique is to send a series of questionnaire by email to a pre-selected group of expert. These questionnaires are designed to elicit and develop individual responses to the questions posed and to enable the experts to refine their views as the groups' work progresses. The main point behind the Delphi method is to overcome the disadvantage of conventional committee actions.

In this case, the task was to reduce the number of options to a manageable number. Fourteen options had been identified for interim upgrades at each plant and twenty-seven options had been identified for built-out to secondary at both plants.

Four individual sewage treatment experts on the project team were asked to independently rate each process. The results of this first evaluation were averaged and the options ranked based on the ratings. The average values derived from all the experts were then presented to them, along with their original rating, for a second round of evaluation. The experts then had the opportunity to revise their ratings in the light of the majority view. The results of the second round evaluation were again averaged and ranked. At this point outlying ratings are identified and the experts came together as a panel to discuss the results. This gives dissenting experts the opportunity to present their reasons for the difference in the rating. This promotes consensus building. The final rating is then completed, and the final ranking is determined.

The summaries showing the results of the above two processes are included in Tables 8.1 to 8.4. Preliminary ranking of processes is shown in Tables 8.5 to 8.8. The minimum and maximum values indicated for each criteria in these tables represent the highest and lowest ranking provided by the panel members.

The panel of experts used was as follows:

- Dr. Bob Dawson, P. Eng. – Stantec Ltd.
- Mr. Harlan Kelly, P. Eng. – Dayton & Knight Ltd.
- Dr. Bill Oldham, P. Eng. – Stantec Ltd.
- Dr. Allan Gibb, P. Eng. – Dayton & Knight Ltd.

The results of the Pass/Fail evaluation and of the Delphi process for selection of options are presented below. These results were reviewed taking account of the specific requirements for treatment at each site. The resulting ranking is logical, specific to each site and to the discharge requirements of the Liquid Waste Management Plan. Further considerations allowed the reduction of options to a manageable number. Careful selection of the combinations to be evaluated for one plant allows for interpolation of results and extrapolation to the other treatment plant.

TABLE 8.1
INITIAL SCREENING IN TASKS 3 AND 4
IIWWTP INTERIM TREATMENT

Process Name	PASS OR FAIL					RATING																					RESULT			RANKING		
	Proven Technology	Discharge Req.	Reliability	Site Suitability	Result	Capital Cost			Operating Cost			Reliability			Integration			Flexibility			Environmental			Social								
						Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
7.2 Physical/Chemical Processes																																
7.2.1 Chemically Enhanced Primary	P	?	P	P	**?**	5.0	5.0	5.0	4.0	2.3	1.0	1.0	1.0	1.0	5.0	5.0	5.0	5.0	3.3	2.0	2.0	1.3	1.0	5.0	4.0	3.0	24.0	21.8	19.0	6	6	5
7.2.2 CEP with Lamella Retrofit to Existing Primaries	P	?	P	P	**?**	5.0	3.5	2.0	3.0	2.3	1.0	2.0	1.3	1.0	5.0	3.0	1.0	5.0	3.3	2.0	2.0	1.5	1.0	5.0	3.5	3.0	19.0	18.3	17.0	11	9	8
7.2.3 CEP with Densedag	P	?		P	**?**	4.0	2.8	2.0	3.0	2.0	1.0	2.0	1.5	1.0	5.0	2.8	1.0	5.0	3.3	2.0	2.0	1.5	1.0	5.0	3.8	3.0	23.0	17.5	14.0	8	10	10
7.2.4 CEP with Ballasted Flocculation Retrofit	P	?	P	P	**?**	4.0	2.8	2.0	3.0	2.0	1.0	2.0	1.5	1.0	5.0	2.8	1.0	5.0	3.3	2.0	2.0	1.5	1.0	5.0	3.8	3.0	23.0	17.5	14.0	8	10	10
7.3 Partial Biological Treatment																																
7.3.2 Conventional Activated Sludge	P	P	P	P	**P**	3.0	2.3	2.0	5.0	3.8	3.0	5.0	4.8	4.0	5.0	4.5	4.0	5.0	4.3	3.0	5.0	3.8	3.0	4.0	3.8	3.0	29.0	27.0	25.0	1	1	1
7.3.3 High Rate Activated Sludge	P	?	P	P	**P**	4.0	3.3	3.0	4.0	3.0	2.0	4.0	3.0	2.0	5.0	4.3	4.0	5.0	3.8	2.0	5.0	2.8	1.0	5.0	3.5	2.0	26.0	23.5	22.0	4	5	3
7.3.4 Roughing or Ultra High Rate Trickling	P	?	P	P	**P**	4.0	3.8	3.0	5.0	4.0	3.0	4.0	3.0	2.0	5.0	4.5	3.0	5.0	3.8	3.0	5.0	3.0	1.0	4.0	2.3	1.0	28.0	24.3	21.0	3	3	4
7.3.5 Trickling Filter Solids Contact	P	P	P	P	**P**	3.0	2.5	2.0	4.0	3.5	3.0	5.0	4.8	4.0	5.0	4.3	3.0	4.0	3.5	2.0	5.0	3.8	2.0	4.0	3.3	2.0	29.0	25.5	18.0	1	2	6
7.3.6 Biological Aerated Filter	P	P	P	P	**P**	3.0	1.8	1.0	4.0	2.8	2.0	5.0	3.5	1.0	4.0	2.0	1.0	5.0	2.5	1.0	5.0	4.0	2.0	5.0	4.8	4.0	24.0	21.3	16.0	6	7	9
7.4 CEP with Partial (25%) Biological	P	?	P	P	**P**	4.0	3.5	3.0	3.0	2.0	1.0	4.0	3.3	3.0	5.0	4.5	4.0	5.0	4.5	4.0	4.0	2.5	1.0	4.0	3.8	3.0	25.0	24.0	23.0	5	4	2
7.5 Dissolved Air Flotation	P	?	P	P	F																											
7.6 Primary Treatment with Add-on Chemical Treatment																																
7.6.1 Chlorination and Dechlorination	P	?	P	P	**?**	5.0	5.0	5.0	3.0	2.0	1.0	3.0	1.8	1.0	5.0	4.0	3.0	4.0	3.3	3.0	1.0	1.0	1.0	3.0	3.0	3.0	23.0	20.0	18.0	8	8	6
7.6.2 Ozonation	P	?	P	P	F																											
7.6.3 Hydrogen Peroxide	P	?	P	P	F																											

TABLE 8.2
INITIAL SCREENING IN TASKS 3 AND 4
IIWWTP BUILD-OUT TO SECONDARY

Process Name	PASS OR FAIL: Proven Technology	Discharge Req.	Reliability	Site Suitability	Result	RATING: Capital Cost			Operating Cost			Reliability			Integration			Flexibility			Environmental			Social			RESULT			RANKING		
						Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
TASK 4: Build Out To Secondary																																
4.1 Fixed Film Processes																																
4.1.1 Roughing Trickling Filter	P	P	P	P	**P**	5.0	4.8	4.0	5.0	4.5	4.0	3.0	1.8	1.0	4.0	3.0	2.0	4.0	3.3	2.0	3.0	2.5	1.0	3.0	2.0	1.0	23.0	21.8	20.0	8	9	7
4.1.2 Standard Rate Trickling Filter	P	P	P	P	**P**	4.0	3.3	3.0	4.0	3.5	3.0	3.0	2.5	1.0	4.0	3.0	2.0	4.0	2.8	2.0	4.0	3.5	3.0	4.0	2.5	2.0	22.0	21.0	20.0	12	11	7
4.1.3 Trickling Filter Solids Contact	P	P	P	P	**P**	4.0	3.3	3.0	4.0	3.3	3.0	5.0	4.8	4.0	5.0	4.0	3.0	5.0	4.0	3.0	5.0	4.5	4.0	5.0	3.0	2.0	32.0	26.8	22.0	1	1	4
4.1.4 Rotating Biological Contactors	F	P	P	F	F																											
4.1.5 Biological Aerated Filter	P	P	P	P	**P**	3.0	1.8	1.0	3.0	2.0	1.0	3.0	3.0	3.0	3.0	2.5	2.0	5.0	3.3	2.0	4.0	3.8	3.0	5.0	4.8	4.0	23.0	21.0	19.0	8	11	10
4.2 Suspended Growth																																
4.2.1 Conventional Activated Sludge	P	P	P	P	**P**	3.0	2.3	1.0	3.0	3.0	3.0	5.0	5.0	5.0	5.0	4.5	4.0	5.0	4.3	3.0	5.0	3.5	2.0	4.0	3.5	2.0	29.0	26.0	23.0	2	2	2
4.2.2 High Rate Activated Sludge	P	P	P	P	**P**	4.0	3.0	1.0	4.0	3.0	2.0	4.0	3.3	2.0	5.0	4.0	3.0	4.0	3.0	2.0	4.0	3.0	2.0	4.0	3.3	2.0	23.0	22.5	21.0	8	7	6
4.2.3 Oxidation Ditch	F	P	P	P	F																											
4.2.4 High Purity Oxygen Activated Sludge	P	P	P	P	**P**	3.0	2.3	1.0	2.0	1.5	1.0	5.0	5.0	5.0	4.0	2.5	1.0	5.0	3.0	2.0	5.0	3.5	2.0	5.0	4.8	4.0	29.0	22.5	19.0	2	7	10
4.2.5 Multi Anoxic Step Feed	P	P	P	P	**P**	1.0	1.0	1.0	5.0	2.8	1.0	5.0	4.8	4.0	5.0	4.3	3.0	5.0	4.5	4.0	5.0	4.3	4.0	4.0	3.5	2.0	27.0	25.0	23.0	4	4	2
4.2.6 Pre-anoxic Activated Sludge	P	P	P	P	**P**	2.0	1.3	1.0	5.0	2.8	1.0	5.0	4.8	4.0	5.0	4.3	3.0	5.0	4.5	4.0	5.0	4.3	4.0	4.0	3.5	2.0	27.0	25.3	24.0	4	3	1
4.2.7 Sequencing Batch Reactor	F	P	P	P	F																											
4.2.8 Membrane Activated Sludge	F	F	P	P	**F**	1.0	1.0	1.0	1.0	1.0	1.0	5.0	4.3	4.0	4.0	2.8	1.0	2.0	1.7	1.0	5.0	3.3	2.0	5.0	4.3	4.0	21.0	18.2	16.0	13	13	13
4.2.9 Deep Shaft Technology (Vertreat)	F	P	P	F	F																											
4.2.10 Upflow Sludge Blanket	F	P	P	F	F																											
4.3 Anaerobic Process																																
4.3.1 CSTR Bioreactor Low Rate	F	F	F	F	F																											
4.3.2 Upflow Anaerobic Sludge Blanket	F	F	F	F	F																											
4.3.3 Packed Bed Filter	F	F	F	F	F																											
4.3.4 Fluidized Bed	F	F	F	F	F																											
4.3.5 Bulk Volume Fermenter	F	F	F	F	F																											
4.3.6 Hybrid Reactor - UASB & Fixed Film	F	F	F	F	F																											
4.4 Fixed Film Suspended Growth																																
4.4.1 Trickling Filter and Activated Sludge	P	P	P	P	**P**	3.0	2.3	2.0	3.0	2.8	2.0	5.0	5.0	5.0	4.0	3.3	3.0	4.0	3.3	3.0	4.0	4.0	4.0	3.0	2.8	2.0	26.0	23.3	22.0	6	6	4
4.4.2 Moving Bed Activated Sludge	F	P	F	?	F																											
4.4.3 Submerged Attached Growth	F	P	F	?	F																											
4.5 Miscellaneous																																
4.5.1 Advanced Oxidation	F	P	F	P	F																											
4.6 Primary Treatment followed by Partial Biological Treatment	P	P	P	P	**P**	5.0	3.5	1.0	4.0	3.3	3.0	3.0	1.8	1.0	5.0	4.0	3.0	4.0	3.3	2.0	5.0	3.5	2.0	5.0	4.3	4.0	25.0	23.5	19.0	7	5	10
4.7 CEP followed by Partial (50%) Biological Treatment	P	?	P	P	**P**	4.0	3.3	3.0	3.0	1.5	1.0	5.0	3.0	1.0	5.0	3.5	1.0	5.0	3.3	2.0	5.0	2.5	1.0	5.0	4.3	3.0	23.0	21.3	20.0	8	10	7

TABLE 8.3
INITIAL SCREENING IN TASKS 3 AND 4
LGWWTP INTERIM TREATMENT

Process Name	PASS OR FAIL					RATING																					RESULT			RANKING		
	Proven Technology	Discharge Req.	Reliability	Site Suitability	Result	Capital Cost			Operating Cost			Reliability			Integration			Flexibility			Environment al			Social								
						Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
LIONS GATE																																
TASK 3: Interim Treatment																																
7.2 Physical/Chemical Processes																																
7.2.1 Chemically Enhanced Primary	P	?	P	P	**?**	5.0	5.0	5.0	3.0	2.0	1.0	1.0	1.0	1.0	5.0	4.8	4.0	4.0	2.8	1.0	3.0	1.5	1.0	5.0	4.3	3.0	23.0	21.3	19.0	6	6	5
7.2.2 CEP with Lamella Retrofit to Existing Primaries	P	?	P	P	**?**	5.0	3.8	2.0	3.0	2.0	1.0	1.0	1.0	1.0	5.0	4.3	2.0	5.0	3.0	1.0	3.0	1.8	1.0	5.0	3.8	3.0	22.0	19.5	16.0	9	8	8
7.2.3 CEP with Densedag	P	?	P	P	**?**	4.0	2.8	2.0	2.0	1.8	1.0	2.0	1.3	1.0	5.0	3.8	3.0	5.0	3.0	1.0	3.0	1.8	1.0	5.0	4.0	3.0	23.0	18.3	16.0	6	9	8
7.2.4 CEP with Ballasted Flocculation Retrofit	P	?	P	P	**?**	4.0	2.8	2.0	2.0	1.5	1.0	2.0	1.3	1.0	5.0	3.5	2.0	5.0	3.0	1.0	3.0	1.8	1.0	5.0	4.0	3.0	23.0	17.8	14.0	6	10	10
7.3 Partial Biological Treatment																																
7.3.2 Conventional Activated Sludge	P	P	P	F	F																											
7.3.3 High Rate Activated Sludge	P	P	P	?	**P**	3.0	2.8	2.0	3.0	2.8	2.0	4.0	3.3	3.0	5.0	3.5	2.0	5.0	3.5	2.0	5.0	4.0	3.0	4.0	3.3	2.0	27.0	23.0	19.0	3	5	5
7.3.4 Roughing or Ultra High Rate Trickling	P	?	P	P	**P**	4.0	3.3	3.0	5.0	4.0	3.0	4.0	3.3	2.0	5.0	3.5	3.0	5.0	3.3	2.0	5.0	3.5	1.0	3.0	2.5	1.0	28.0	23.3	20.0	1	4	4
7.3.5 Trickling Filter Solids Contact	P	P	P	?	**P**	3.0	2.3	2.0	4.0	3.3	3.0	5.0	4.8	4.0	4.0	3.8	3.0	4.0	3.8	3.0	5.0	4.5	4.0	4.0	3.0	2.0	28.0	25.3	22.0	1	2	2
7.3.6 Biological Aerated Filter	P	P	P	P	**P**	3.0	2.3	2.0	4.0	2.8	2.0	5.0	4.0	3.0	5.0	4.3	4.0	5.0	3.5	2.0	5.0	4.0	3.0	5.0	5.0	5.0	27.0	25.8	25.0	3	1	1
7.4 CEP with Partial (25%) Biological	P	?	P	P	**P**	4.0	3.0	2.0	3.0	2.0	1.0	4.0	3.5	3.0	5.0	4.3	4.0	5.0	4.0	3.0	4.0	3.0	2.0	4.0	3.8	3.0	26.0	23.5	21.0	5	3	3
7.5 Dissolved Air Flotation	P	F	P	P	F																											
7.6 Primary Treatment with Add-on Chemical Treatment																																
7.6.1 Chlorination and Dechlorination	P	?	P	P	**?**	5.0	5.0	5.0	3.0	2.0	1.0	1.0	1.0	1.0	5.0	5.0	5.0	3.0	2.3	1.0	2.0	1.5	1.0	3.0	3.0	3.0	20.0	19.8	19.0	10	7	5
7.6.2 Ozonation	P	?	P	P	F																											
7.6.3 Hydrogen Peroxide	P	?	P	P	F																											

TABLE 8.4
INITIAL SCREENING IN TASKS 3 AND 4
LGWWTP BUILD-OUT TO SECONDARY

Process Name	PASS OR FAIL					RATING																					RESULT			RANKING		
	Proven Technology	Discharge Req.	Reliability	Site Suitability	Result	Capital Cost			Operating Cost			Reliability			Integration			Flexibility			Environmental			Social								
						Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
TASK 4: Build Out To Secondary																																
4.1 Fixed Film Processes																																
4.1.1 Roughing Trickling Filter	P	P	P	P	**P**	5.0	4.8	4.0	5.0	4.5	4.0	3.0	1.8	1.0	4.0	3.0	2.0	4.0	3.0	2.0	5.0	2.5	1.0	3.0	1.8	1.0	23.0	21.3	17.0	10	9	10
4.1.2 Standard Rate Trickling Filter	P	P	P	F	F																											
4.1.3 Trickling Filter Solids Contact	P	P	P	?	**P**	4.0	3.3	3.0	4.0	3.8	3.0	5.0	5.0	5.0	5.0	4.0	3.0	5.0	3.8	2.0	5.0	4.0	3.0	5.0	3.0	2.0	32.0	26.8	23.0	1	1	3
4.1.4 Rotating Biological Contactors	P	P	P	F	F																											
4.1.5 Biological Aerated Filter	P	P	P	P	**P**	3.0	2.8	2.0	3.0	2.5	2.0	5.0	4.3	4.0	4.0	3.5	3.0	5.0	3.5	2.0	5.0	4.3	3.0	5.0	5.0	5.0	27.0	25.8	25.0	5	3	1
4.2 Suspended Growth																																
4.2.1 Conventional Activated Sludge	P	P	P	F	F																											
4.2.2 High Rate Activated Sludge	P	P	P	?	**P**	4.0	4.0	4.0	4.0	3.0	2.0	5.0	3.3	2.0	5.0	4.0	3.0	5.0	4.3	3.0	3.0	2.3	1.0	4.0	3.3	2.0	28.0	24.0	22.0	4	5	4
4.2.3 Oxidation Ditch	P	P	P	F	F																											
4.2.4 High Purity Oxygen Activated Sludge	P	P	P	?	**P**	5.0	2.8	1.0	2.0	1.3	1.0	5.0	5.0	5.0	5.0	3.5	1.0	3.0	2.8	2.0	5.0	3.5	2.0	5.0	4.5	4.0	29.0	23.3	19.0	2	6	8
4.2.5 Multi Anoxic Step Feed	?	P	P	F	F																											
4.2.6 Pre-anoxic Activated Sludge	P	P	P	F	F																											
4.2.7 Sequencing Batch Reactor	P	P	P	?	**?**	3.0	2.8	2.0	4.0	2.8	2.0	5.0	4.0	3.0	4.0	3.3	3.0	4.0	3.3	3.0	4.0	3.5	3.0	4.0	2.8	1.0	24.0	22.3	20.0	7	7	7
4.2.8 Membrane Activated Sludge	F	F	P	P	**F**	1.0	1.0	1.0	1.0	1.0	1.0	5.0	4.3	4.0	5.0	3.3	1.0	4.0	2.8	1.0	4.0	3.0	2.0	5.0	4.5	4.0	24.0	19.8	16.0	7	11	11
4.2.9 Deep Shaft Technology (Vertreat)	P	P	P	P	**P**	2.0	1.5	1.0	3.0	2.8	2.0	4.0	3.3	3.0	4.0	3.3	3.0	3.0	2.0	1.0	4.0	3.8	3.0	5.0	4.5	4.0	24.0	21.0	19.0	7	10	8
4.2.10 Upflow Sludge Blanket	P	P	P	F	F																											
4.3 Anaerobic Process																																
4.3.1 CSTR Bioreactor Low Rate	F	F	F	F	F																											
4.3.2 Upflow Anaerobic Sludge Blanket	F	F	F	F	F																											
4.3.3 Packed Bed Filter	F	F	F	F	F																											
4.3.4 Fluidized Bed	F	F	F	F	F																											
4.3.5 Bulk Volume Fermenter	F	F	F	F	F																											
4.3.6 Hybrid Reactor - UASB & Fixed Film	F	F	F	F	F																											
4.4 Fixed Film Suspended Growth																																
4.4.1 Trickling Filter and Activated Sludge	P	P	P	?	**P**	3.0	2.8	2.0	4.0	3.3	3.0	5.0	5.0	5.0	4.0	3.3	2.0	5.0	3.8	2.0	4.0	4.0	4.0	3.0	2.5	2.0	26.0	24.5	22.0	6	4	4
4.4.2 Moving Bed Activated Sludge	F	P	F	P	F																											
4.4.3 Submerged Attached Growth	F	P	F	P	F																											
4.5 Miscellaneous																																
4.5.1 Advanced Oxidation	F	P	F	P	F																											
4.6 Primary Treatment followed by Partial Biological Treatment	P	P	P	P	**P**	5.0	4.8	4.0	4.0	3.3	3.0	3.0	1.8	1.0	5.0	4.5	4.0	5.0	3.8	2.0	4.0	3.8	3.0	5.0	4.3	4.0	29.0	26.0	25.0	2	2	1
4.7 CEP followed by Partial (50%) Biological Treatment	P	?	P	P	**P**	4.0	3.5	3.0	3.0	1.5	1.0	5.0	3.0	2.0	5.0	4.0	3.0	5.0	3.5	3.0	4.0	2.3	1.0	5.0	4.3	4.0	23.0	22.0	21.0	10	8	6

NOTES:

P = Pass, F = Fail

? = Information inadequate for decision. Where research is in progress to establish the results the process is passed, otherwise the process is failed.

P (Bold) = Process proceeds to next round of evaluation

? (Bold) = Awaiting results of GVRD testwork. Process proceeds to Ranking

F (Bold) = Process has future promise. Process proceeds to Ranking

TABLE 8.5
IIWWTP PRELIMINARY RANKING – INTERIM TREATMENT
RESULTS SUMMARY

PROCESS NAME		RESULTS	RANKING
IONA ISLAND			
TASK 3: Interim Treatment Option			
7.2.2	Conventional Activated Sludge	27.0	**1**
7.2.4	Trickling Filter Solids Contact	25.5	**2**
7.2.3	Roughing or Ultra High Rate Trickling Filter	24.3	**3**
7.3	CEP with Partial (25%) Biological Treatment	24.0	**4**
7.2.3	High Rate Activated Sludge	23.5	**5**
7.1.1	Chemically Enhanced Primary	21.8	**6**
7.2.5	Biologically Aerated Filter	21.3	**7**
7.5.1	Chlorination and Dechlorination	20.0	**8**
7.1.2	CEP with Lamella Retrofit to Existing Primaries	18.3	**9**
7.1.3	CEP with DensaDeg	17.5	**10**
7.1.4	CEP with Ballasted Flocculation Retrofit	17.5	**10**

TABLE 8.6
IIWWTP PRELIMINARY RANKING – BUILD OUT TO SECONDARY
RESULTS SUMMARY

PROCESS NAME	RESULTS	RANKING
IONA ISLAND		
TASK 4: Build Out To Secondary		
4.1.3 Trickling Filter Solids Contact	26.8	**1**
4.2.1 Conventional Activated Sludge	26.0	**2**
4.2.6 Pre-anoxic Activated Sludge	25.3	**3**
4.2.5 Multi Anoxic Step Feed	25.0	**4**
4.6 Primary Treatment followed by Partial Biological Treatment	23.5	**5**
4.4.1 Trickling Filter and Activated Sludge	23.3	**6**
4.2.2 High Rate Activated Sludge	22.5	**7**
4.2.4 High Purity Oxygen Activated Sludge	22.5	**7**
4.1.1 Roughing Trickling Filter	21.8	**9**
4.7 CEP followed by Partial (50%) Biological Treatment	21.3	**10**
4.1.2 Standard Rate Trickling Filter	21.0	**11**
4.1.5 Biological Aerated Filter	21.0	**11**
4.2.8 Membrane Activated Sludge	18.2	**13**

TABLE 8.7
LGWWTP PRELIMINARY RANKING – INTERIM TREATMENT
RESULTS SUMMARY

PROCESS NAME		RESULTS	RANKING
LIONS GATE			
TASK 3: Interim Treatment Option			
7.2.5	Biologically Aerated Filter	25.8	**1**
7.2.4	Trickling Filter Solids Contact	25.3	**2**
7.3	CEP with Partial (25%) Biological Treatment	23.5	**3**
7.2.4	Roughing or Ultra High Rate Trickling Filter	23.3	**4**
7.2.3	High Rate Activated Sludge	23.0	**5**
7.1.1	Chemically Enhanced Primary	21.3	**6**
7.5.1	Chlorination and Dechlorination	19.8	**7**
7.1.2	CEP with Lamella Retrofit to Existing Primaries	19.5	**8**
7.1.3	CEP with DensaDeg	18.3	**9**
7.1.4	CEP with Ballasted Flocculation Retrofit	17.8	**10**

TABLE 8.8
LGWWTP PRELIMINARY RANKING – BUILD OUT TO SECONDARY
RESULTS SUMMARY

PROCESS NAME	RESULTS	RANKING
IONA ISLAND		
TASK 4: Build Out to Secondary		
4.1.3 Trickling Filter Solids Contact	26.8	**1**
4.6 Primary Treatment followed by Partial Biological Treatment	26.0	**2**
4.1.5 Biological Aerated Filter	25.8	**3**
4.4.1 Trickling Filter and Activated Sludge	24.5	**4**
4.2.2 High Rate Activated Sludge	24.0	**5**
4.2.4 High Purity Oxygen Activated Sludge	23.3	**6**
4.2.7 Sequencing Batch Reactor	22.3	**7**
4.7 CEP followed by Partial (50%) Biological Treatment	22.0	**8**
4.1.1 Roughing Trickling Filter	21.3	**9**
4.2.9 Deep Shaft Technology (Vertreat)	21.0	**10**
4.2.8 Membrane Activated Sludge	19.8	**11**

8.2 IONA ISLAND WWTP

8.2.1 Delphi Ranking

8.2.1.1 Interim Treatment

1. Conventional Activated Sludge
2. Trickling Filter Solids Contact
3. Roughing Trickling Filter
4. Chemically Enhanced Primary with Partial Biological Treatment
5. High Rate Activated Sludge
6. Chemically Enhanced Primary

8.2.1.2 Build-out to Secondary

1. Trickling Filter Solids Contact
2. Conventional Activated Sludge
3. Pre-anoxic activated sludge
4. Multi-anoxic step feed
5. Primary Treatment followed by Biological Treatment of part of the flow
6. Trickling Filter and Activated Sludge
7. High Rate Activated Sludge
8. High Purity Oxygen Activated Sludge

8.2.1.3 Comments

1. Pre-anoxic activated sludge and Multi-anoxic step feed are nutrient removal processes and provide a level of treatment not required for this project. They should therefore be excluded from consideration. However, both processes are capable of reducing final clarifier size – the first by providing a selector zone that can improve settleability, and the second by reducing the solids loading on the clarifiers

2. Since primary treatment exists at the plants most options would be preceded by primary treatment. Use of Chemically Enhanced Primary treatment could provide a flexible level of treatment, which could allow the size of the secondary plant to be reduced. This should be evaluated as a pre-treatment option.

3. The extent of biological treatment provided at build-out time, after Chemically Enhanced Primary treatment, would be just sufficient to treat the reduced load.

4. Because interim standards do not require high levels of suspended solids removal, trickling filters, appropriately loaded and without solids contact, may be used for interim treatment.

Taking these comments into account, results in an interim ranking:

8.2.2 Interim Ranking:

8.2.2.1 Interim Treatment

1. Primary Treatment followed by Conventional Activated Sludge
2. Primary Treatment followed by Roughing Trickling Filter
3. High Rate Activated Sludge
4. Chemically Enhanced Primary

Chemically Enhanced Primary treatment should be considered as a pre-treatment except for High Rate Activated Sludge

8.2.2.2 Build-out to Secondary

1. Primary Treatment followed by Trickling Filter Solids Contact
2. Primary Treatment followed by Conventional Activated Sludge
3. Primary Treatment followed by Trickling Filter and Activated Sludge
4. High Rate Activated Sludge in parallel with Primary Treatment
5. Primary Treatment followed by High Purity Oxygen Activated Sludge

Chemically Enhanced Primary treatment should be considered as a pre-treatment.

Progression from interim to Secondary: High Rate Activated Sludge interim treatment would need to be configured for upgrading to conventional activated sludge. Use of High Purity Oxygen Activated Sludge would require adoption in interim treatment if it is preferred.

8.2.3 Progressing to Final Ranking:

8.2.3.1 Interim Treatment

In order to reduce the number of options to a manageable number, the following options were evaluated. The combination allows the cost of various levels of treatment to be determined and for the interpolation of various different options.

1A. Primary Treatment followed by 50% ADWF Conventional Activated Sludge
1B. Primary Treatment followed by 100% ADWF Conventional Activated Sludge
2. Primary Treatment followed by 50% ADWF Roughing Trickling Filter
3. 50% ADWF High Rate Activated Sludge in parallel with Primary Treatment
4. Chemically Enhanced Primary followed by 50% RTF
5. Chemically Enhanced Primary applied to all flow.

8.2.3.2 Build-out to Secondary

The following processes are a logical progression from the interim processes:

1. Primary Treatment followed by 2 x ADWF Trickling Filter Solids Contact
2A. Primary Treatment followed by 2 x ADWF Conventional Activated Sludge
2B. Primary Treatment followed by 2 x ADWF Conventional Activated Sludge including flow from the Lions Gate Plant
3. Chemically Enhanced Primary Treatment followed by 60% of 2 x ADWF Conventional Activated Sludge

8.3 LIONS GATE WWTP

8.3.1 Delphi Ranking:

8.3.1.1 Interim Treatment

1. Biological Aerated Filter
2. Trickling Filter Solids Contact
3. Chemically Enhanced Primary with Partial Biological Treatment
4. Roughing Trickling Filter
5. High Rate Activated Sludge
6. Chemically Enhanced Primary

8.3.1.2 Build-out to Secondary

1. Trickling Filter Solids Contact
2. Primary Treatment Followed by Partial Biological Treatment
3. Biological Aerated Filter
4. Trickling Filter and Activated Sludge
5. High Rate Activated Sludge
6. High Purity Oxygen Activated Sludge
7. Sequencing Batch Reactor
8. Chemically Enhanced Primary with Partial Biological Treatment

8.3.1.3 Comments

1. Since primary treatment exists at the plant, most options would be preceded by primary treatment.

2. Use of Chemically Enhanced Primary treatment could provide a flexible level of treatment, which could allow the size of the secondary plant to be reduced. This should be evaluated as a pre-treatment option.

3. The extent of biological treatment provided, at build-out time, after Chemically Enhanced Primary treatment would be just sufficient to treat the reduced load.

4. Because interim standards do not require high levels of suspended solids removal, trickling filters, appropriately loaded and without solids contact, may be used for interim treatment

Taking these comments into account reduces the ranking list of the following:

8.3.2 Interim Ranking:

8.3.2.1 Interim Treatment

1. Primary Treatment followed by Biological Aerated Filter
2. Primary Treatment followed by Roughing Trickling Filter
3. High Rate Activated Sludge
4. Chemically Enhanced Primary

Chemically Enhanced Primary treatment should be considered as a pre-treatment except for High Rate Activated Sludge

8.3.2.2 Build-out to Secondary

1. Primary Treatment followed by Trickling Filter Solids Contact
2. Primary Treatment Followed by Biological Aerated Filter
3. Primary Treatment Followed by Trickling Filter and Activated Sludge
4. High Rate Activated Sludge
5. Primary Treatment Followed by High Purity Oxygen Activated Sludge
6. Primary Treatment Followed by Sequencing Batch Reactor

Chemically Enhanced Primary treatment should be considered as a pre-treatment except for High Rate Activated Sludge.

Progression from Interim to Secondary would require adoption in interim treatment of High Purity Oxygen Activated Sludge or Sequencing Batch Reactors if these are preferred.

8.3.3 Progressing to Final Ranking

8.3.3.1 Interim Treatment:

In order to reduce the number of options to a manageable number, the following options were evaluated. The combination allows the cost of various levels of treatment to be determined and for the interpolation of various different options.

1. 50% ADWF Biological Aerated Filter in parallel with Primary Treatment
2A. 50% ADWF Roughing Trickling Filter in parallel with Primary Treatment
2B. 100% ADWF Roughing Trickling Filter in parallel with Primary Treatment
3. Chemically Enhanced Primary followed by 50% ADWF Roughing Trickling Filter
4. 50 % ADWF High Rate Activated Sludge in parallel with Primary Treatment

8.3.3.2 Build-out to Secondary

The following processes are a logical progression from the interim processes:

1. Primary Treatment followed by 2 x ADWF Trickling Filter Solids Contact,
2. Primary Treatment Followed by 2 x ADWF Biological Aerated Filter,
3. 2 x ADWF High Rate Activated Sludge in parallel with Primary Treatment,
4. Chemically Enhanced Primary Treatment followed by 60% of 2 x ADWF Trickling Filter Solids Contact.

9 DETAILED ANALYSIS OF OPTIONS THAT PASSED FIRST LEVEL OF SCREENING

To make the results of the analysis of different processes comparable, a set of textbook process design parameters were adopted. These were applied to the analysis of the unit operations in each plant using the demand flows and loads.

Because the textbook process design parameters may differ from those experienced in the treatment plants, the analysis results may be at variance with practical experience. Once the options that passed the first round of screening have been compared and reduced to a short list, parameters experienced in the plants will be utilized for the final analysis which is reported in Appendix 10. The required upgrades and the timing and cost thereof will be reported.

The following methodology was used to analyze the interim upgrade options:

- Preliminary process design was used for the built-out options. The worksheets for process design are included in Appendix A.
- Flows and load projections developed in Section 4. Upper and lower envelopes were developed and the upper envelope was used as the basis for process design. Flows and load projections included separate projections for residential, commercial and institutional, industrial, trucked liquid waste and inflow and infiltration. The
- From the results of the process design, the size and number of unit processes was estimated. This is summarized in Sections 9.1.3 for Iona Island and 9.2.3 for Lions Gate
- Based on the number and size of each unit processes, conceptual site layout were developed.
- For all upgrades options, estimated sludge productions were calculated as well as energy requirements.
- Preliminary capital cost estimates were then developed. Preliminary capital cost estimates for the build-out options are summarized in Section 9.1.6 for Iona Island and in Section 9.2.6 for Lions Gate. Further details on the cost estimates are included in Appendix B.
- Preliminary operating and maintenance cost estimates were developed and these incorporated sludge handling cost and energy cost. See section 9.1.7 for Iona Island and 9.2.7 for Lions Gate.

Following the second level screening, a short list of interim upgrade options was developed. Following review comments from GVRD, the flow and load projections were modified to develop a design case that falls between the lower and upper envelope. The above procedure was repeated for the short list of options only and the results are detailed in Appendix 10. In essence, Appendix 10 is the continuation of this Appendix 3

9.1 IONA ISLAND

9.1.1 General

Conventional activated sludge (CAS), roughing trickling filter (RTF), and high-rate activated sludge (HRAS) are the three biological processes that passed the first level of process screening and that were considered for IIWWTP interim treatment upgrades. Chemical enhanced primary (CEP) only, or CEP followed by partial biological treatment with RTF, was also considered as an option to reduce the capacity of secondary biological treatment. The following six (6) upgrade options were developed with varied design capacities and interim treatment process:

- Option 1A: Primary + 50% average dry weather flow (ADWF) CAS
- Option 1B: Primary + 100% ADWF CAS
- Option 2: Primary + 50% ADWF RTF
- Option 3: 50% ADWF HRAS + (Q – 50% ADWF) Primary
- Option 4: CEP + 50% ADWF RTF
- Option 5: CEP only.

Further analyses of these upgrade options are detailed in this section prior to proceeding with the second level of process screening. Brief process descriptions, schematic flow diagrams, conceptual process designs including plant layouts, footprint requirements, sludge productions, effluent quality projections, capital and O&M (operating and maintenance) cost estimates, process flexibility and other factors are discussed in the followings sections. Following discussions at Workshop # 3, the construction of additional primary sedimentation tanks was deleted from all upgrade options.

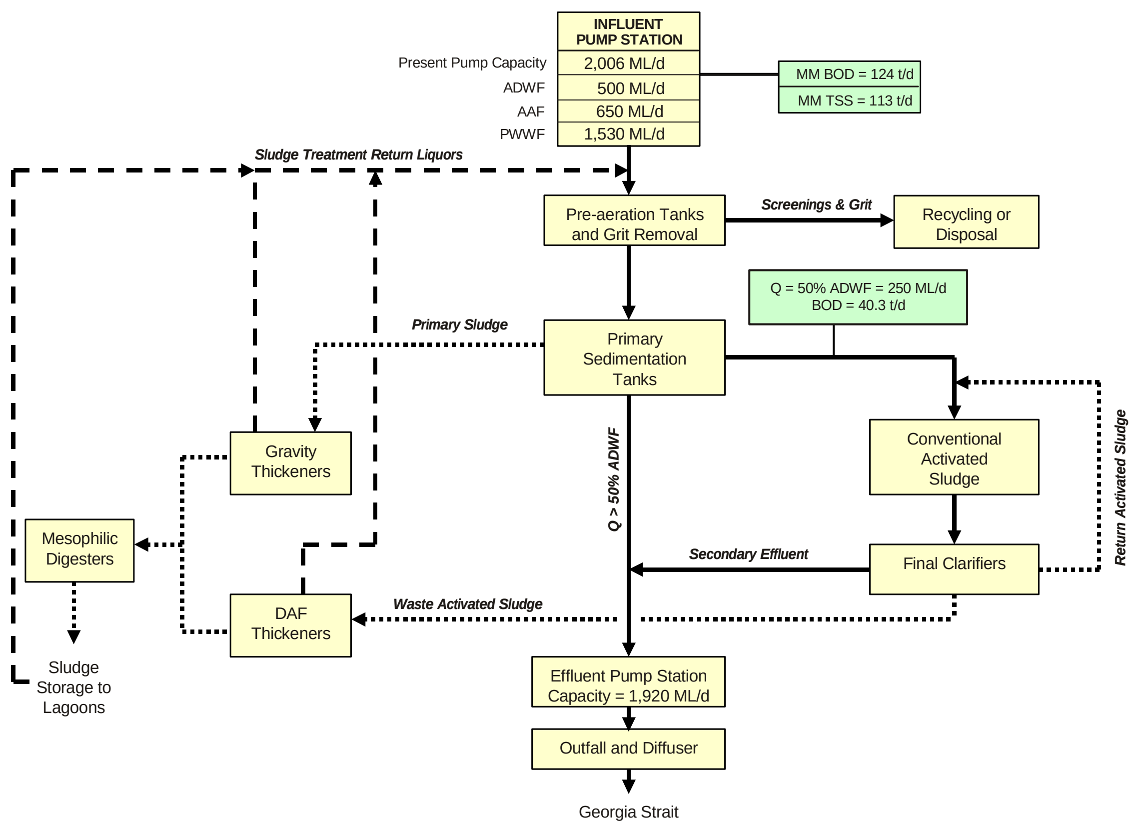
9.1.2 Description of Upgrade Options

9.1.2.1 Option 1A: Primary + 50% ADWF CAS

The preliminary (screen and grit removal) and primary (primary sedimentation tank) treatment units are designed to treat the entire peak wet weather flow collected from the Vancouver Sewage Area (VSA). The CAS process is designed to provide treatment for 50% of average dry weather flow (ADWF) for a hydraulic capacity of 50% x 500 ML/d = 250 ML/d, at 50% of the maximum month (MMF) loading of 40 t/d of BOD and 29 t/d of TSS after the primary treatment units. Final clarifiers will be used to remove TSS and biological sludge generated from the CAS process. The portion of the flow greater than 50% of the ADWF will bypass the secondary treatment units. The primary treated flow and the portion of the flow that receives biological treatment would be combined and discharged directly to the outfall pump station. A process schematic of this option is illustrated in Figure 9.1.

The primary sludge and the biological sludge are thickened in the gravity thickeners and dissolved air flotation (DAF) units, respectively. The thickened sludge from both streams will be stabilized in the same anaerobic digesters to achieve volatile solids reduction and pathogen kills. The anaerobic digesters are designed to operate at mesophilic condition during the interim stage, with the design capability to be operated at thermophilic condition for future expansion (e.g. build-out to secondary). The digested biosolids will then be placed in the adjacent drying lagoon for dewatering and storage. The supernatant from the lagoons will be returned to the process for treatment. The dry solids will be stockpiled onsite or be transported to other land application or storage sites. Because of the capacity limitation of the sludge lagoon, mechanical dewatering may be necessary at some point. Interim sludge handling is discussed in more detail in Appendix 7.

FIGURE 9.1
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 1A

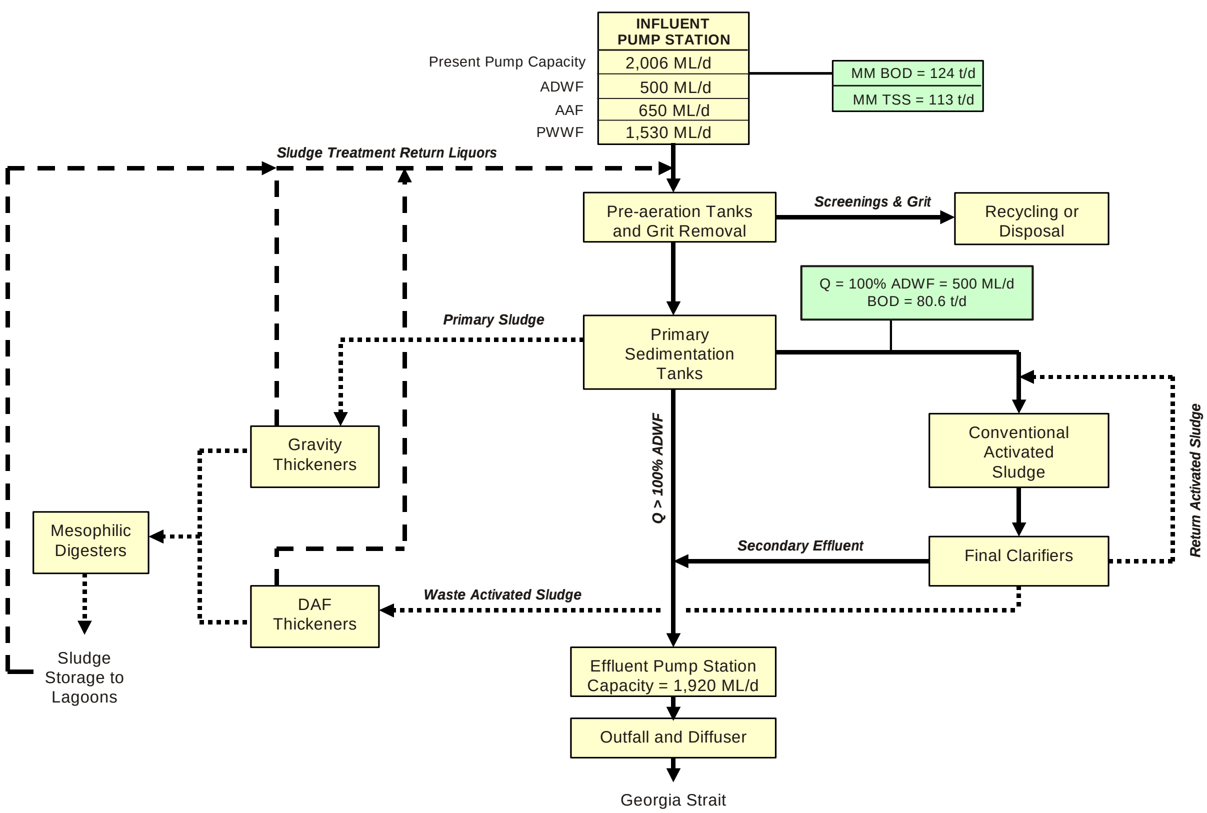


9.1.2.2 <u>Option 1B: Primary + 100% ADWF CAS</u>

This option is basically the same as option 1A except with a different design flow and loads for the biological treatment process. The rationale of increasing the design flow for biological treatment from 50% to 100% of ADWF was based on the results of the small-scale testing which indicate the portion of flow that should receive biological treatment in order to achieve improved toxicity test results during dry weather condition. The results indicated that a minimum of 50%~75% ADWF flow should receive biological treatment in order to achieve this goal. CAS will be applied as the secondary treatment. A process schematic is illustrated in Figure 9.2. The design flow and loads of the biological process are 100% of ADWF (500 ML/d) and 100% of MMF loadings after primary treatment (81 t/d of BOD and 57 t/d of TSS). Arrangements for solids handling and reject wastewater treatment are the same as in Option 1A.

By treating 100% of ADWF, the footprint requirement, capital investment, O&M costs will be significantly higher than treating only 50% of ADWF. However, the effluent quality and effluent toxicity reduction can be improved substantially, particularly under dry weather conditions.

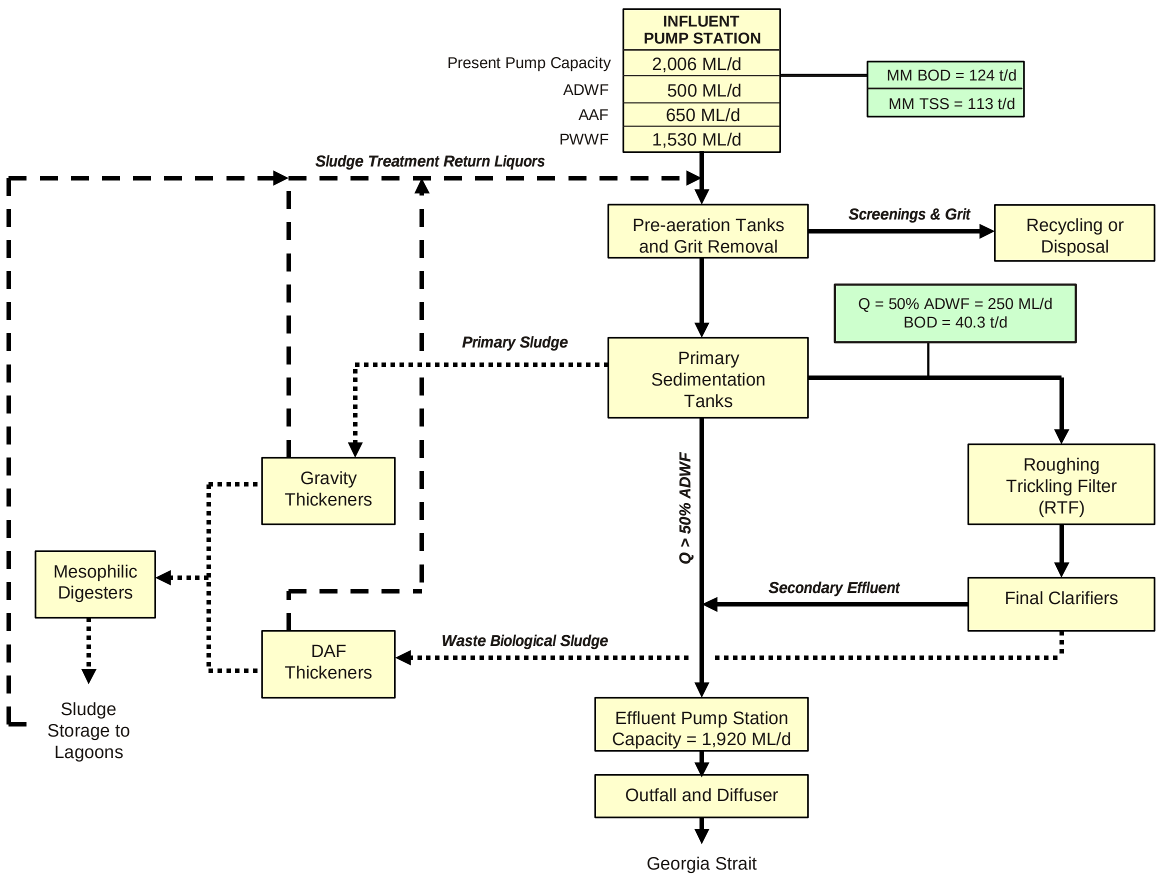
FIGURE 9.2
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 1B



9.1.2.3 Option 2: Primary + 50% ADWF RTF

This option is similar to option 1A except that RTF would be used for biological treatment. A process schematic is illustrated in Figure 9.3. The entire sewage flow will be treated by the preliminary and primary units, followed by the RTF process to treat 50% of the ADWF (50% x 500 ML/d = 250 ML/d) and 50% of MMF loadings (40 t/d of BOD and 29 t/d of TSS after primary treatment). Final clarifiers will be used after the RTF units to remove TSS and biological sludge from the filter slough off. The portion of the flow greater than 50% of the ADWF will bypass the secondary treatment units. The primary treated flow and the portion of the flow that receives biological treatment would be combined and discharged directly into the outfall pump station. The arrangements for solids handling and reject wastewater treatment are the same as in Option 1A.

FIGURE 9.3
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 2



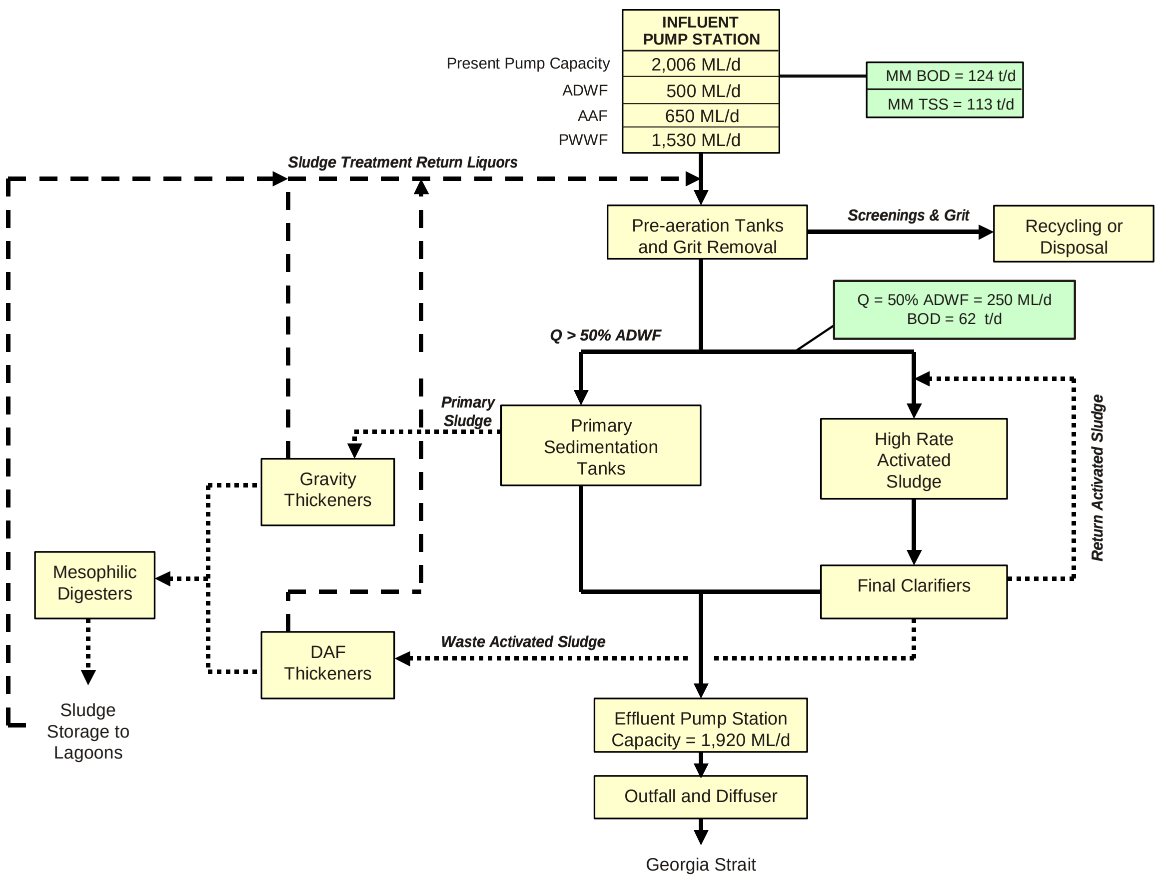
9.1.2.4 Option 3: 50% ADWF HRAS + (Q – 50% ADWF) Primary

In this option, HRAS will be used in conjunction with primary treatment. Figure 9.4 shows a process schematic with the HRAS and primary treatment processes in parallel. Following preliminary screening and grit removal, 50% of ADWF

(50% x 500 ML/d = 250 ML/d) will be directed to the HRAS process for treatment. The remaining flow (greater than 250 ML/d) will be treated in the primary units only. The primary and HRAS effluents will be combined together before final discharge to the outfall.

TSS and the biological sludge from the HRAS units will be removed in the final clarifiers and thickened in the DAF units. Sludge from the primary sedimentation tanks will be pumped to the gravity thickeners for thickening. The arrangement of solids handling and reject wastewater treatment are the same as in Option 1A.

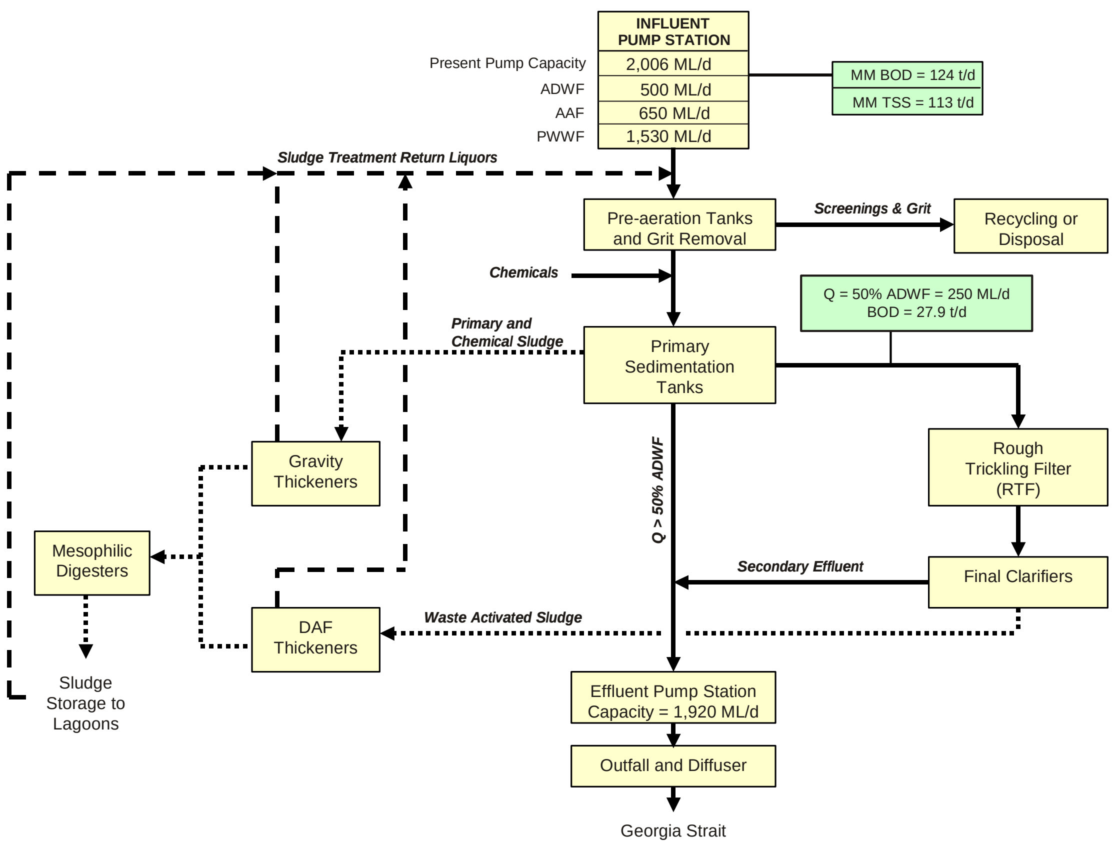
FIGURE 9.4
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 3



9.1.2.5 Option 4: CEP + 50% ADWF RTF

This option is a modification of Option 2 with the addition of chemical enhanced primary (CEP) treatment prior to partial biological treatment using RTF. Chemicals (alum and polymer) are added prior to the primary sedimentation tanks in order to increase TSS and BOD removal efficiency. A process schematic is shown in Figure 9.5. Similar to Option 2, the entire flow will receive preliminary and CEP treatment. Following the CEP process, 50% of the ADWF (50% x 500 ML/d = 250 ML/d) will undergo the RTF process (or CAS and HRAS). The design loadings in the RTF process are 50% MMF loadings, i.e. 28 t/d of BOD and 12 t/d of TSS. Final clarifiers will be provided after the RTF units to remove TSS and biological sludge. The portion of the flow greater than 50% of the ADWF will bypass the secondary treatment units. The CEP treated flow and the portion of the flow that receives biological treatment would be combined and discharged directly to the outfall pump station. The combined primary and chemical sludge will be collected in the primary sedimentation tanks and thickened in the gravity thickeners. The biological sludge from the secondary treatment process will be thickened in the DAF thickeners. The solids handling arrangement subsequent to the thickeners is the same as in Option 1A.

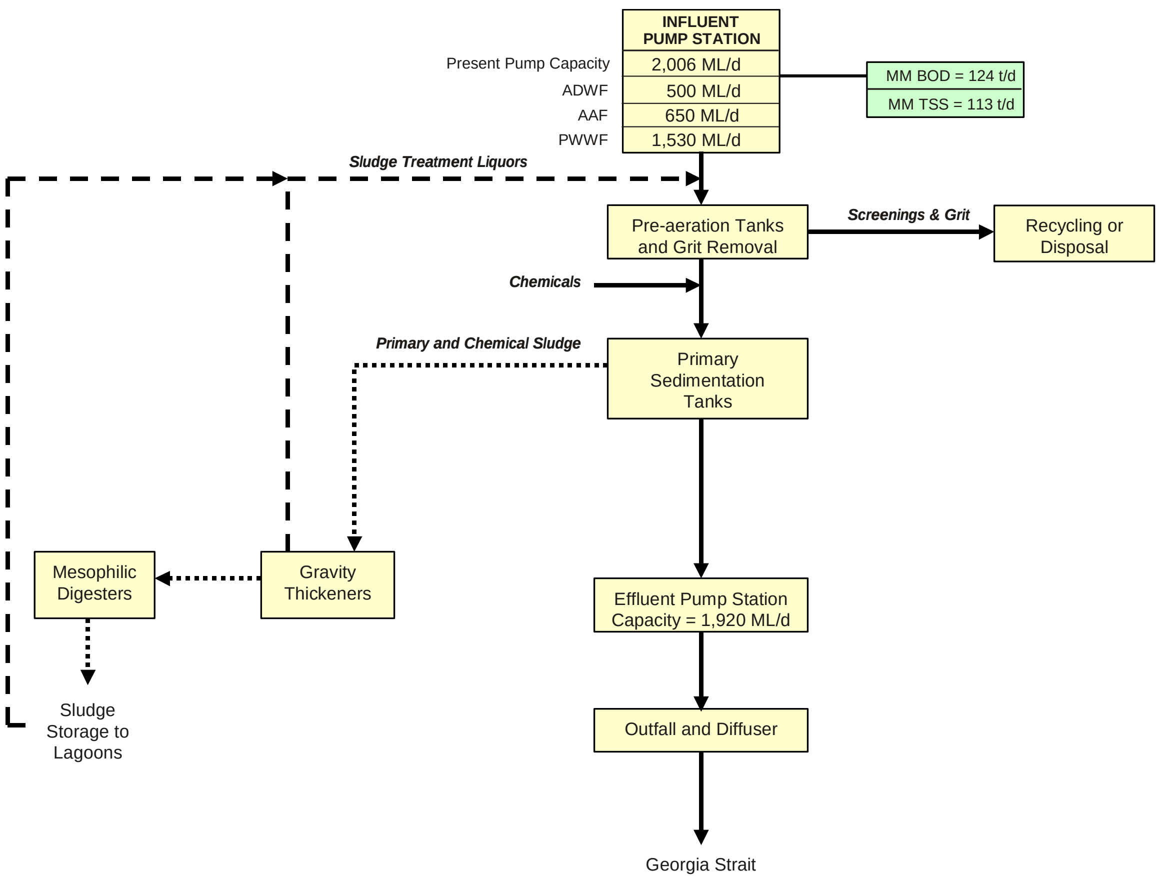
FIGURE 9.5
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 4



9.1.2.6 Option 5: CEP only

This option involves only the addition of chemicals to the existing primary process in order to achieve enhanced TSS and BOD removals As shown in the process schematic in Figure 9.6, the entire flow and loads will be treated by the preliminary and CEP units. Combined primary and chemical sludge will be collected and thickened in the gravity thickeners. The thickened sludge will be stabilized in the anaerobic digesters. The digested sludge will then be dewatered in the adjacent lagoons, and the dry solids will be stockpiled onsite or other land application sites. The rejected wastewater from the sludge handling processes (thickeners, digesters and dewatering units) is recycled back to the system for treatment.

FIGURE 9.6
PROCESS SCHEMATIC OF IIWWTP INTERIM UPGRADE OPTION 5



9.1.3 Tank Size and Number of Unit Required

A spreadsheet model was developed to carry out the conceptual process design and to determine the number of process units required for each upgrade option. The model summary is included in Appendix A with the details. The unit process dimensions and number of units required are summarized in Tables 9.1 and 9.2 respectively. The actual unit dimensions are adjusted slightly in each option to obtain the integral number of units presented in Table 9.2. To eliminate the primary sedimentation tank expansion in Option 3 (where only one unit is required), its primary process will probably be overloaded during the PWWF condition.

Using typical design values for the SOR for the primary sedimentation tanks indicated that additional primary sedimentation tanks would be needed. However, this issue was re-examined using actual performance of the primary sedimentation tanks. This is further discussed in Appendix 10. Using actual performance instead of textbook values and increasing the size of the secondary treatment units to deal with the increase in loading would eliminate the need for additional primary sedimentation tanks.

The DAF thickeners are designed based on operation without the addition of polymer. The anaerobic digesters are designed based on single-stage complete-mixed mesophilic operation with a minimum HRT of 20 days. If the digesters were designed for operation under thermophilic conditions, the minimum HRT would be reduced to 15 days and the digester volume could be reduced by approximately 33%. This should be considered in conjunction with the needs of future build-out to secondary expansion.

The conceptual site layouts for each upgrade option are illustrated in Figures 9.7, 9.8, 9.9, 9.10, 9.11, and 9.12. A modular concept is proposed for the ease of development from the interim treatment stage to future expansion for the build out to secondary. New primary and secondary clarifiers, DAF, and dewatering units are located on land owned by the GVS&DD east of the existing plant. New digesters and gravity thickeners are located in the south and southeast corners of existing plant site, respectively. An addition to the administration building and control room can be built by extending the southeast wing of existing building. Chemical storage and associated facilities can be constructed at the existing location subject to requirements for preloading in the case of very high tanks.

For comparison purposes, the approximate total footprint requirements of each upgrade option are shown in Table 9.3 (actual reactors/building footprint plus 20% for spacing between tanks, piping galleries, roads and miscellaneous structures. Based on a design biological treatment capacity of 250 ML/d (50% of ADWF), Option 1A (CAS process) requires the largest space, followed by Option 3 (HRAS process) and Option 2 (RTF process). Because of the additional removals resulting from CEP, Option 4 requires shorter RTF tanks than Option 2. However, the footprint required for Option 4 is about 20% larger than Option 2 because of the additional thickeners and digesters need to deal with increased sludge production. Option 5 which consists of CEP only requires the smallest footprint for the expansion of primary and associated sludge handling units. Depending on the option, 8% to 40% of the land owned by the GVS&DD east of existing plant (estimated about 130,000 m^2 in total) will be required for interim upgrading.

A forecast of effluent quality for TSS and BOD for the period from 2004 to 2020 was carried out and is included in Appendix 10.

TABLE 9.1
IIWWTP UNIT PROCESS DIMENSIONS FOR EACH UPGRADE OPTION

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Primary Sedimentation Tank						
Length (m)	66.0	66.0	66.0	66.0	66.0	66.0
Width (m)	15.0	15.0	15.0	15.0	15.0	15.0
Depth (m)	2.7	2.7	2.7	2.7	2.7	2.7
Unit Size (m^2)	990	990	990	990	990	990
Existing Surface area (m^2)	11,819	11,819	11,819	11,819	11,819	11,819
Total Area Required (m^2)	16,250	16,250	16,250	12,800	16,250	16,250
Aeration/Solids Contact Tank						
Length (m)	86.0	86.0		86.0		
Width (m)	30.0	30.0		30.0		
Depth (m)	5.5	5.5		5.5		
Unit Reactor Volume (m^3)	14,190	14,190		14,190		
Total Volume Required (m^3)	55,413	110,825		35,819		
Roughing Trickling Filter (RTF)						
Diameter (m)			44.0		44.0	
Depth (m)			4.6		3.2	
Total Volume Required (m^3)			11,514		7,971	
Final Clarifier						
Diameter (m)	44.0	44.0	44.0	44.0	44.0	
Depth (m)	4.5	4.5	4.5	4.5	4.5	
Unit Size (m^2)	1,520	1,520	1,520	1,520	1,520	
Total Area Required (m^2)	5,556	11,111	3,472	5,667	3,472	
Gravity Thickener						
Diameter (m)	19.8	19.8	19.8	19.8	19.8	19.8
Depth (m)	3.0	3.0	3.0	3.0	3.0	3.0
Unit Size (m^2)	308	308	308	308	308	308
Existing Surface area (m^2)	616	616	616	616	616	616
Total Area Required (m^2)	565	565	565	283	1,056	1,056
DAF Thickener						
Diameter (m)	20.0	20.0	20.0	20.0	20.0	
Depth (m)	3.5	3.5	3.5	3.5	3.5	
Unit Size (m^2)	314	314	314	314	314	
Total Area Required (m^2)	552	1103	576	1249	367	
Digester						
Diameter (m)	32.0	32.0	32.0	32.0	32.0	32.0
Depth (m)	10.6	10.6	10.6	10.6	10.6	10.6
Unit Size (m^3)	8,521	8,521	8,521	8,521	8,521	8,521
Existing Volume (m^3)	19,816	19,816	19,816	19,816	19,816	19,816
Total Volume Required (m^3)	37,729	52,857	38,396	45,557	52,283	42,227

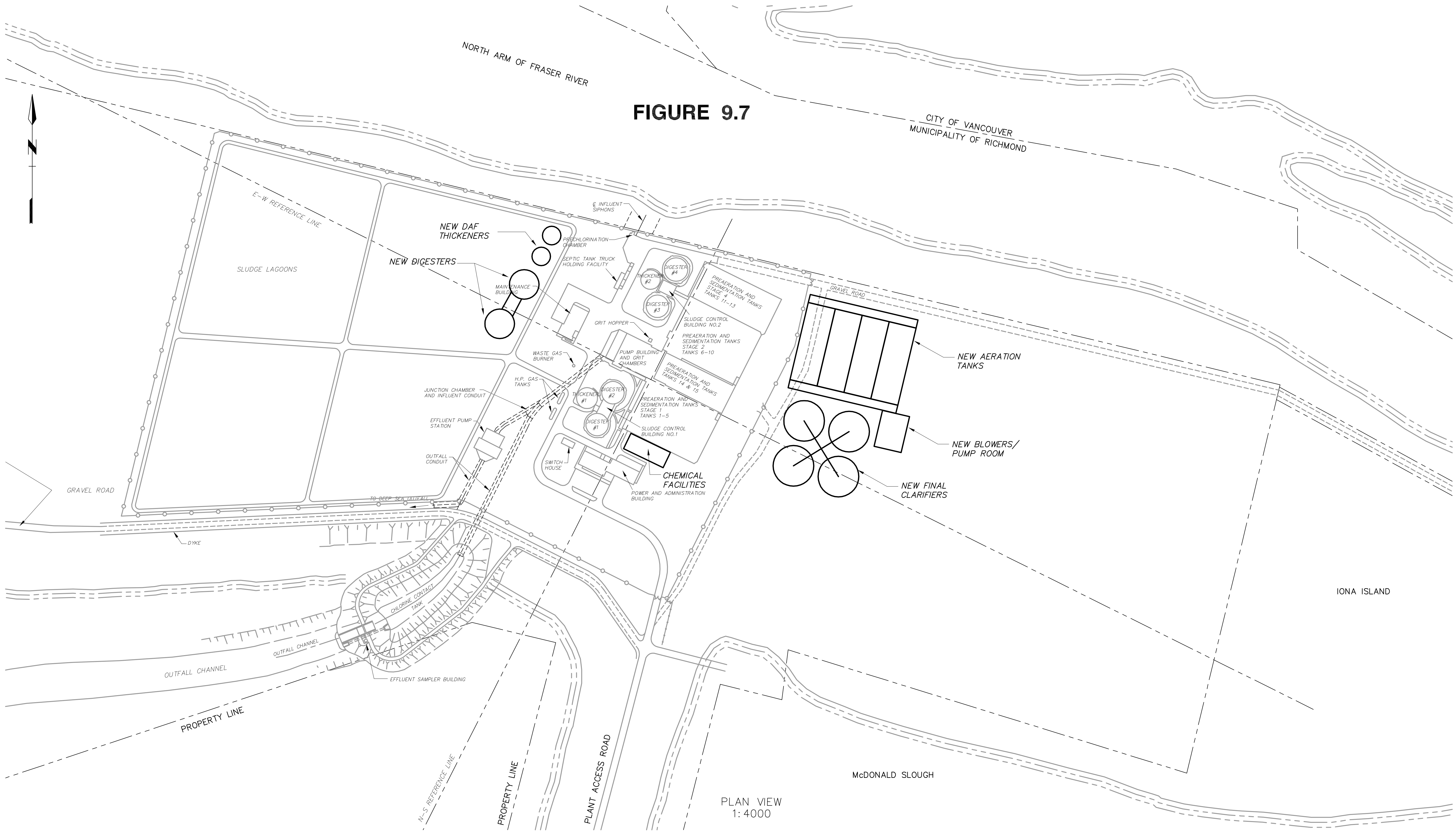
TABLE 9.2
IIWWTP NUMBER OF UNITS OF REQUIRED FOR EACH UPGRADE OPTION

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Primary Sedimentation Tank						
Total Requirement	20	20	20	15	20	20
Existing	15	15	15	15	15	15
Addition	5	5	5	0	5	5
Aeration/Solids Contact Tank						
Total Requirement	4	8		3		
Existing	0	0		0		
Addition	4	8		3		
Roughing Trickling Filter						
Total Requirement			2		2	
Existing			0		0	
Addition			2		2	
Final Clarifier						
Total Requirement	4	8	3	4	3	
Existing	0	0	0	0	0	
Addition	4	8	3	4	3	
Gravity Thickener						
Total Requirement	2	2	2	1	4	4
Existing	2	2	2	2	2	2
Addition	0	0	0	0	2	2
DAF Thickener						
Total Requirement	2	4	2	4	1	
Existing	0	0	0	0	0	
Addition	2	4	2	4	1	
Digester						
Total Requirement	6	8	6	7	8	7
Existing	4	4	4	4	4	4
Addition	2	4	2	3	4	3

TABLE 9.3
IIWWTP FOOTPRINT REQUIREMENTS FOR EACH UPGRADE OPTION

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Total Footprint Required (m^2)	30,600	52,200	20,000	22,500	22,700	9,600

FIGURE 9.7



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December 10, 2003 11:38 a.m.

Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103

ACAD DWG. INTERIM-1A 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chk'd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

INTERIM

OPTION 1A
PRIMARY + 50% ADWF CAS

SCALE: AS SHOWN

DISTRICT FILE

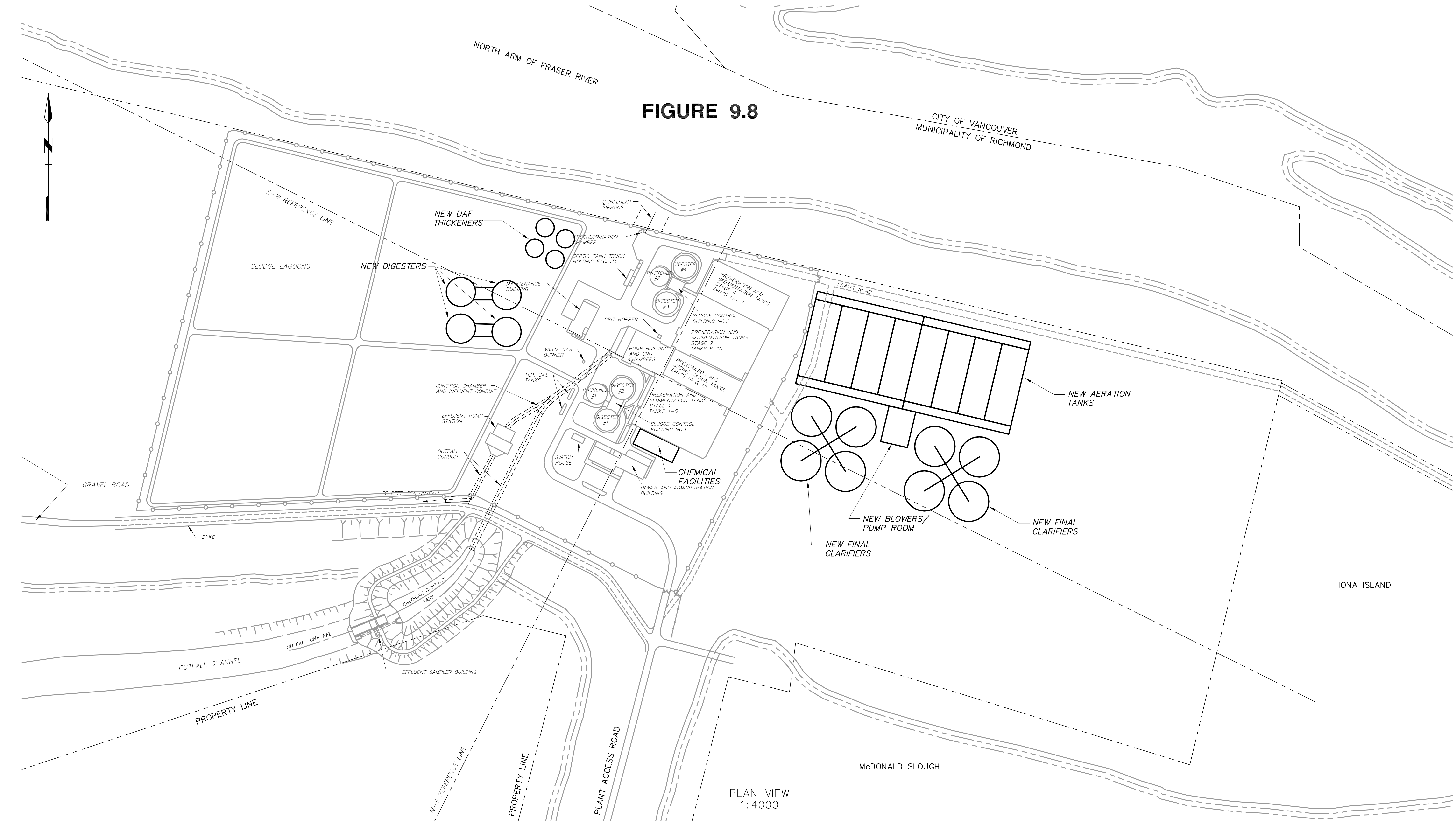
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG:

SAROS DWG. ID No.:

FIGURE 9.8



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Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103

ACAD DWG. INTERIM-1B 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chk'd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

INTERIM
OPTION 1B
PRIMARY + 100% ADWF CAS

SCALE: AS SHOWN

DISTRICT FILE

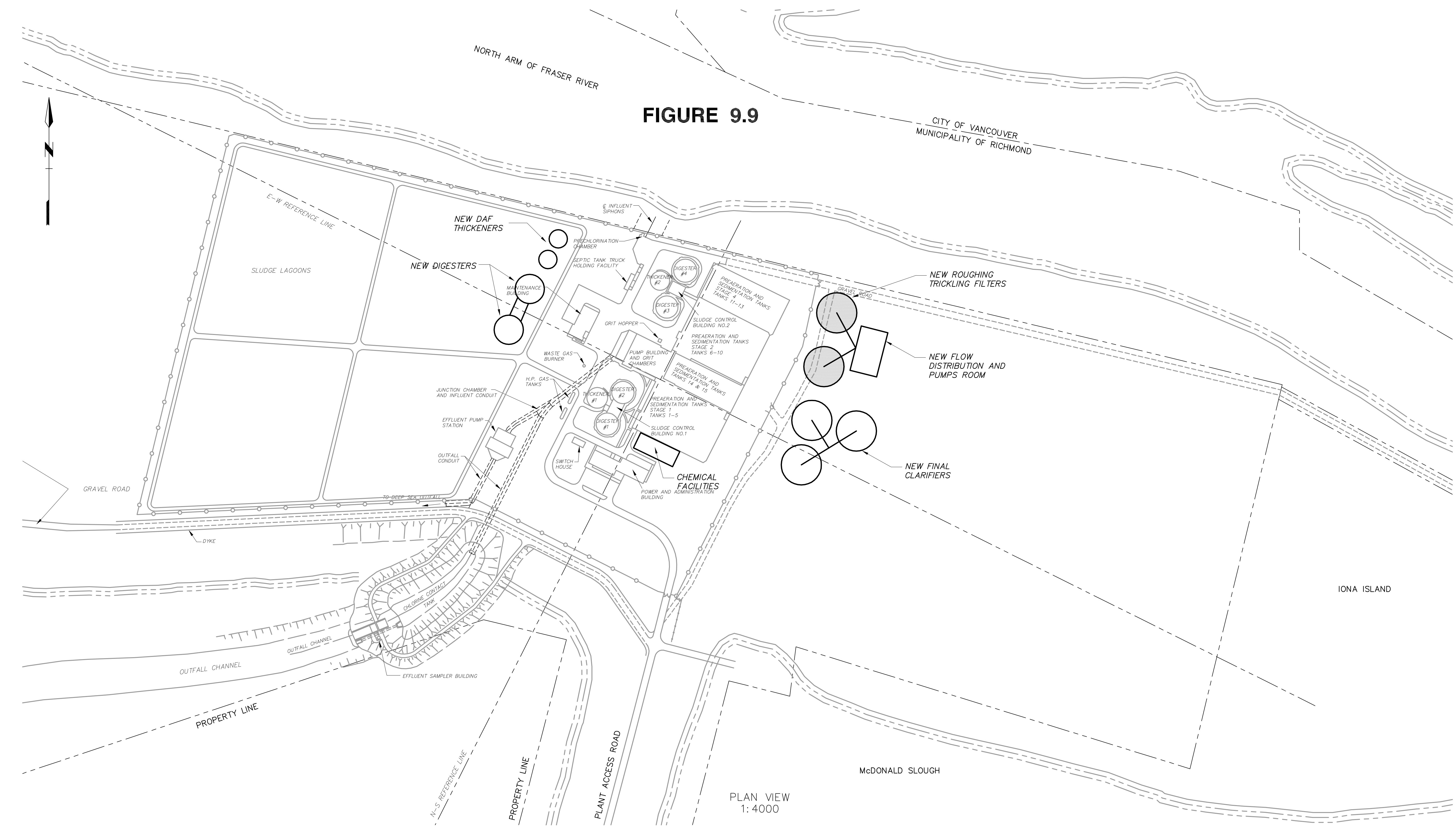
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG:

SAROS DWG. ID No.:

FIGURE 9.9



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December 03, 2003 1:40 p.m.

Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103

ACAD DWG. INTERIM-2 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

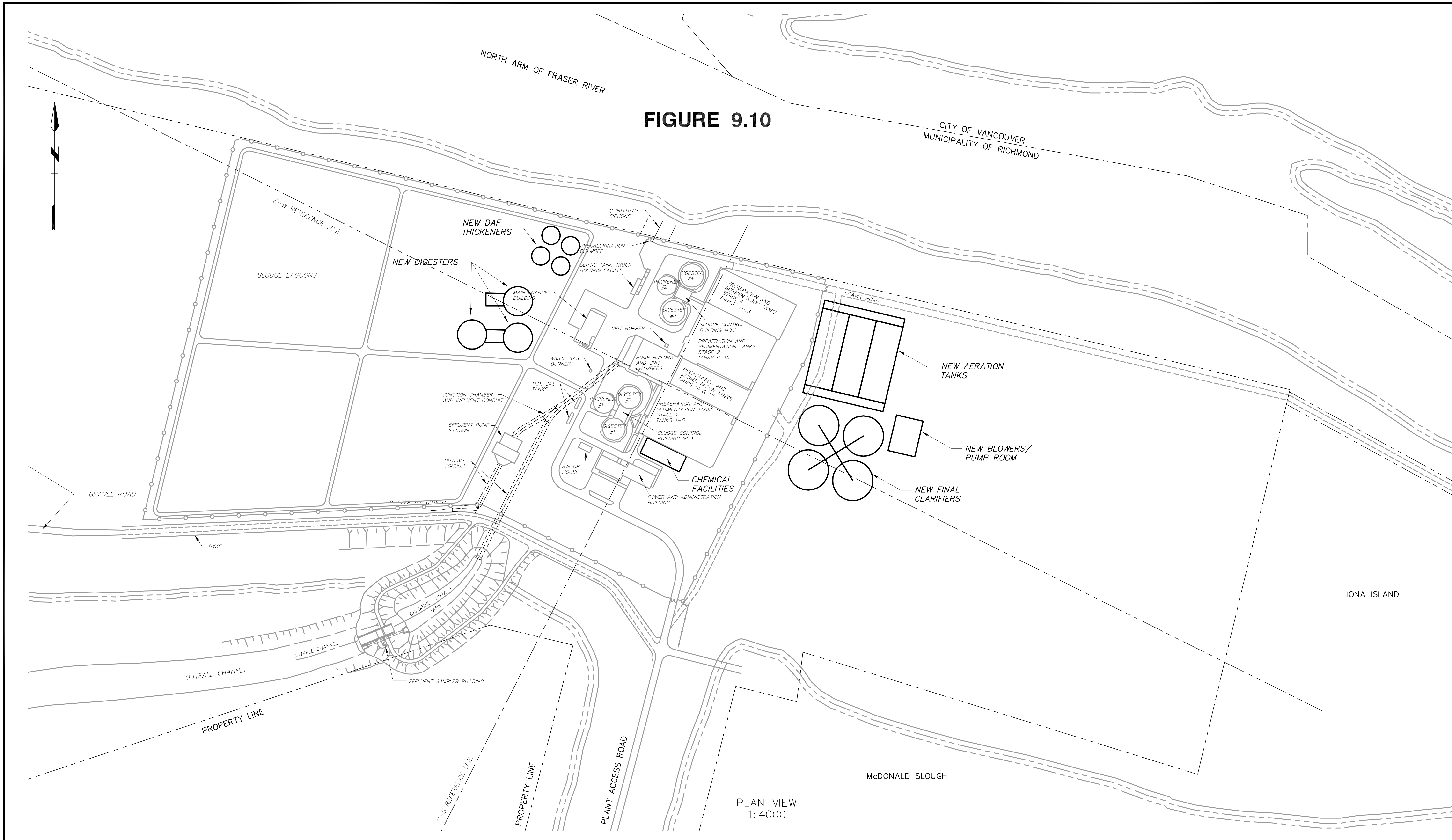
INTERIM
OPTION 2
PRIMARY + 50% ADWF RTF

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG:
SAROS DWG. ID No.:

FIGURE 9.10



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Dayton & Knight Ltd.

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DWG. No. 415-1-103

ACAD DWG. INTERIM-3 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

INTERIM

OPTION 3
50% ADWF HRAS + (Q – 50% ADWF)
PRIMARY

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO ➡

XREF DWG:

SAROS DWG. ID No.:

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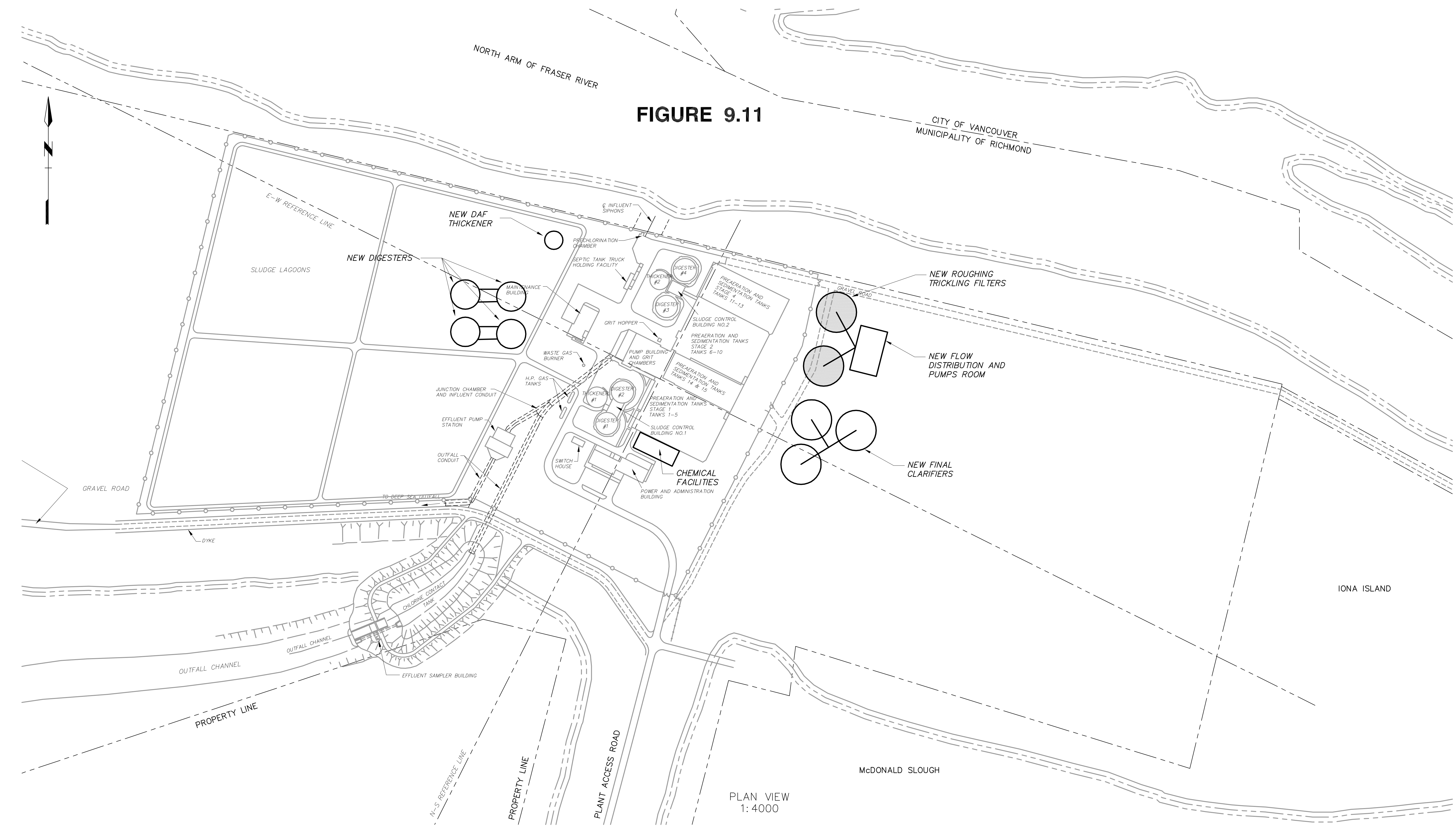
FIGURE 9.11



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Stantec

DWG. No. 415-1-103 ACAD DWG. INTERIM-4 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

INTERIM

OPTION 4
CEP + 50% ADWF RTF

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG: SAROS DWG. ID No.:

Xrefs: Option.DWG;
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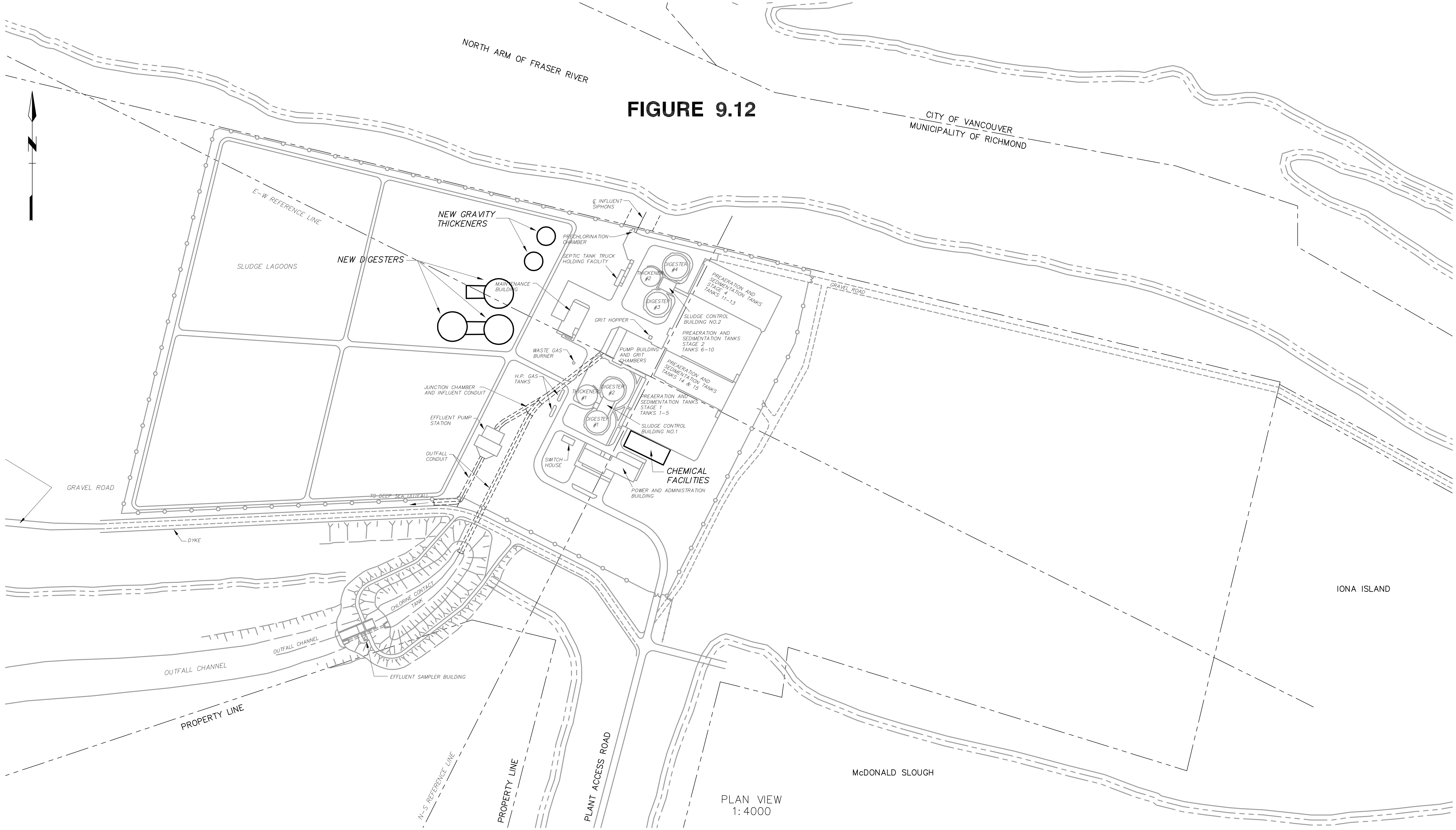
December 02, 2003 3:18 p.m.

FIGURE 9.12



Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103 ACAD DWG. INTERIM-5 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chk'd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

INTERIM
OPTION 5
CEP ONLY

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG:
SAROS DWG. ID No.:

9.1.4 Projected Effluent Quality

Projected effluent qualities (BOD and TSS concentrations) for each upgrade option are shown in Table 9.4. The projected quality is based on the annual average flow condition (1.3 x ADWF). Essentially, the biological process for Options 1A, 2, 3, and 4 would be operated at their maximum hydraulic design capacity all year around, since the design AAF of 650 ML/d is significantly larger than 50% of ADWF (250 ML/d). With the same design treatment capacity, Option 1A (CAS process), Option 2 (RTF process) and Option 3 (HRAS process) will achieve the same levels of effluent quality, which satisfy the current permit requirements for BOD and TSS concentrations. Option 4 can be expected to achieve even better effluent quality due to the enhanced solids removal of CEP followed by partial biological treatment. Option 1B with 100% of ADWF biological treatment capacity can provide the greatest BOD removals. Option 5 with CEP only can achieve the same level of BOD and better TSS quality than the other 50% partial biological treatment options (i.e. Option 1A, Option 2, and Option 3).

TABLE 9.4
IIWWTP EFFLUENT CONCENTRATION PROJECTIONS OF EACH UPGRADE OPTION

YEAR	**2002 Average**	**2021 Interim**					
Option		Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Design AAF (MLD)	574	650	650	650	650	650	650
Effluent BOD (mg/L)	84	62	36	63	56	44	61
Effluent SS (mg/L)	49	52	32	52	48	26	29

9.1.5 Sludge Production Projections

The projected sludge production for the maximum month loading for each upgrade option is shown in Table 9.5 for primary, chemical and biological sludge respectively. The sludge quantities are expressed in dry solids. The estimated sludge volumes at various sludge handling stages are also shown in Table 9.5, including the gravity thickener underflow, DAF supernatant, digested sludge and dewatered sludge. It should be noted that the sludge generation rates indicated in Table 9.5 are based on the maximum monthly loading. These values were used to size the solids handling unit processes. The increase in sludge production compared to current averages of 970 m^3/d of raw/digested sludge and 84 m^3/d dewatered sludge at 35% solids concentration is summarized in Table 9.6 for their dry weight and bulk volumes. The solids content of the digested sludge and dewatered sludge concentrations are estimated about 2.7~3.6% and 27~35%, respectively.

For the same design capacity, Option 4 is expected to produce the greatest sludge volumes due to the combination of CEP and biological (RTF) processes. Option 1A, Option 2, and Option 3 will produce equivalent amount of sludge on a dry ton basis. Option 5 which consists of CEP only will produce more sludge than the partial biological treatment options, but less than Option 4. It should to be noted that chemical sludge is less digestible in the stabilization process (e.g. anaerobic digestion), and less beneficial for reuse because of lower available nutrient content (e.g. agriculture and land application). Further discussion of interim sludge handling is provided separately (Appendix 7).

TABLE 9.5
IIWWTP SLUDGE PRODUCTION
FOR EACH INTERIM UPGRADE OPTION (MAXIMUM MONTH)

YEAR		**2021 Interim**					
Option	Unit	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Raw Sludge/Biosolids							
Primary Sludge	T/d	57	57	57	28	-	-
CEP Sludge	T/d	0	0	0	0	106	106
Secondary Biosolids	T/d	26	53	28	60	18	0
Total Raw Sludge	T/d	83	109	84	88	123	106
Thickened Sludge							
Gravity Thickener	m^3/d	1130	1130	1130	565	2111	2111
DAF Supernatant	m^3/d	756	1513	790	1713	503	0
Total Thickened Sludge	m^3/d	1886	2643	1920	2278	2614	2111
Digested Sludge	m^3/d	1886	2643	1920	2278	2614	2111
Dewatered Sludge	m^3/d	215	279	196	229	246	215

TABLE 9.6
IIWWTP INCREASE OF SLUDGE COMPARED TO CURRENT LEVEL BASED ON AVERAGE ANNUAL LOADING

YEAR		**2021 Interim**					
Option	Unit	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Raw Sludge	%	71	126	73	82	154	118
Thickened Sludge	%	94	172	98	135	170	118
Digested Sludge	%	94	172	98	135	170	118
Dewatered Sludge	%	156	232	134	172	193	155

9.1.6 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 9.7. Detailed breakdowns of the cost estimates are included in Appendix B. Option 5, which consists of CEP only, has the lowest capital expenditure. All capital cost estimates are expressed in 2003 dollars. Capital costs for the various unit processes have been estimated on the basis of cost curves developed from construction and estimated cost on projects in Western Canada. The costs used in the cost curves are adjusted using the ENR index. The cost curves for the various unit processes include equipment, tankage, mechanical, electrical and building required to house the equipment.

For the options with partial biological treatment for 50% of ADWF, the RTF option has the lowest capital cost, followed by the CAS and HRAS options. However the difference in capital cost between these three options is not significant. Option 4, with a combination of CEP and biological treatment using RTF, has a higher capital cost than the other partial biological options, mainly due to the additional facilities required to handle the large volume of sludge. However, the difference in capital cost between Option 4 and the three options with 50% ADWF biological treatment is not significant.

For the interim period, it was initially assumed that the current solids handling method of lagoon storage followed by on-site stockpiling will continue and as a result the cost of mechanical dewatering was not included in the capital cost estimates. As discussed in Appendix 7 and 10, one of the four sludge storage lagoon will be required for interim plant expansion. Mechanical dewatering has been included in the updated cost estimates included in Appendix 10. It should be noted that the capital cost estimates for the interim options as presented in this report were developed to provide sufficient information for the second level of screening. Following the second level of screening, a short list of options was developed. This short list of options was analyzed in more detail in Appendix 10 and included updated cost estimates.

TABLE 9.7
IIWWTP CAPITAL COSTS OF EACH INTERIM UPGRADE OPTION

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
CAPITAL COSTS						
Site Improvements	$17,195,000	$23,400,000	$17,195,000	$18,360,000	$17,195,000	$11,270,000
Primary Sedimentation Tank	$0	$0	$0	$0	$0	$0
Chemical Feed	$0	$0	$0	$0	$1,500,000	$1,500,000
Aeration Basin/Solids Contact	$20,434,000	$40,867,000	$0	$15,325,000	$0	$0
Roughing Trickling Filter	$0	$0	$11,619,000	$0	$8,288,000	$0
Secondary Clarifiers	$13,011,000	$26,022,000	$9,758,000	$13,011,000	$9,758,000	$0
Gravity Thickeners	$0	$0	$0	$0	$2,772,000	$2,772,000
DAF Thickeners	$11,693,000	$23,386,000	$11,982,000	$26,627,000	$7,988,000	$0
Digesters	$16,836,000	$31,056,000	$17,619,000	$24,026,000	$30,433,000	$20,823,000
Site Works	$2,793,000	$5,630,000	$1,680,000	$2,793,000	$2,523,000	$1,116,000
Control System	$2,479,000	$4,853,000	$4,500,000	$3,160,000	$2,370,000	$944,000
Electrical substation	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000	$1,500,000
Odour Control	$0	$0	$1,000,000	$0	$1,000,000	$0
Division 1 Cost	$1,719,000	$3,333,000	$1,513,000	$2,161,000	$1,703,000	$716,000
Engineering	$13,751,000	$25,074,000	$12,431,000	$16,768,000	$13,652,000	$6,388,000
Project Management/QA/QC	$3,438,000	$6,269,000	$3,108,000	$4,192,000	$3,413,000	$1,597,000
Contingency	$25,782,000	$47,015,000	$23,309,000	$31,441,000	$25,598,000	$11,977,000
Sub-Total	$130,631,000	$238,406,000	$118,057,000	$159,364,000	$129,694,000	$60,604,000
Net GST, 0% of Sub-Total	$0	$0	$0	$0	$0	$0
Total Capital Costs	$130,631,000	$238,406,000	$118,057,000	$159,364,000	$129,694,000	$60,604,000

9.1.7 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs (November 2003 dollars) of each upgrade option at 2020 flows are shown in Table 9.8. The existing primary plant has a staff of 57. For options 1A, 2, 3 and 5 it is estimated that the staff would increase to 65 persons. For option 1B, which provides treatment for 100% of ADWF, it is estimated the staff would increase to 70 persons. For the combination of CEP and biological treatment, it is also estimated that the staff would increase to 70 persons. The chemical costs for options 4 and 5 include alum and polymer. No chemical disinfection (e.g. chlorination) is required during interim operation.

The residual management costs are estimated based on a rate of $100/tonne for hauling, reuse (e.g. land application), and other fixed expenses, assuming that land application sites are available. The solids concentration is estimated to be about 75% if the current practice of lagoon storage following by on-site stockpiling continues. It is assumed that the stockpiled solids will be hauled away on a yearly basis. If mechanical dewatering using a centrifuge were provided, the solids contents would be 30~35% and the cost of residual management would double as a result of the larger volume.

TABLE 9.8
IIWWTP OPERATING AND MAINTENANCE COSTS OF EACH INTERIM UPGRADE OPTION

YEAR	2021 Interim					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Labour	$4,359,000	$4,695,000	$4,359,000	$4,359,000	$4,695,000	$4,695,000
Chemical Costs	$0	$0	$0	$0	$8,600,000	$8,600,000
Residuals Management	$3,272,000	$4,242,000	$2,986,000	$3,478,000	$3,746,000	$3,266,000
Energy/Power	$2,121,000	$3,200,000	$1,171,000	$1,909,000	$1,247,000	$1,162,000
Repair/Maintenance	$3,821,000	$4,683,000	$3,720,000	$4,051,000	$3,814,000	$3,261,000
Administration and others	$1,469,000	$1,287,000	$1,469,000	$1,469,000	$1,234,000	$1,440,000
Total (O&M Costs)	$15,042,000	$18,108,000	$13,706,000	$15,266,000	$23,337,000	$22,425,000

The above costs include the 2002 operating and maintenance costs of $9,091,000. Costs are in present (November 2003) values.

9.1.8 Life Cycle Cost

The preliminary life cycle costs (LCC) of each upgrade option are estimated in Table 9.9 at present values in 2003 dollars, using 6% of discount rate and the period from 2004 to 2020 as the planning horizon. The assumptions made in the life cycle cost analysis are as follows:

- Construction of interim upgrades in 2006 and 2007
- Operating cost for 2004 to 2007 based on primary plant
- Operating cost for 2008 to 2020 based on primary plant plus interim upgrade.

Option 1B has the highest LCC at present worth, followed by Option 4. Options 1A, 2, 3 and 5 have similar net present worth.

TABLE 9.9
IIWWTP LIFE CYCLE COST ESTIMATE OF EACH INTERIM UPGRADE OPTIONS

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q 50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
O&M Costs at present value	$136,108,000	$157,608,000	$126,740,000	$137,679,000	$194,274,000	$187,879,000
Discounted Capital Costs	$106,575,000	$194,505,000	$96,318,000	$130,018,000	$105,812,000	$49,444,000
Total Capital and O&M Costs at present value	$242,683,000	$352,113,000	$223,058,000	$267,697,000	$300,086,000	$237,323,000

9.1.9 Flexibility of Phasing

Plant development could be facilitated by phasing in modular expansions from interim to build-out to secondary. Taking the CAS option as an example, the unit process could be expanded from one module (250 ML/d) during the interim and four modules (1,000 ML/d) in the build-out to secondary stage. Extending the height of RTF units and adding extra units could be used to expand the RTF option during the interim. The capacity could then be expanded to meet the design capacity in build-out to secondary using the TF/SC process. Additional sludge handing units can be phased in when needed. All capital investment made during the interim stage can be utilized at the build-out to secondary stage.

9.1.10 Energy Requirements

The major energy requirement for operating the secondary process is power for pumping and aeration. The existing influent and effluent pumps are still necessary to transport and discharge wastewater. Additional pumps are required to increase the hydraulic gradient after the primary process (primary effluent) and ensure the gravity flow through out the bioreactors, final clarifiers, to the effluent pump chamber. Extra pump power is needed in the RTF option to elevate the primary effluent to the top of RTF tower. Aeration power is essential for the CAS and HRAS bioreactors and additional pumps are required to serve internal recycling (e.g. return activated sludge) and the sludge handling operation (e.g. wasted activated sludge, scum collection, DAF, and anaerobic digesters).

Table 9.10 presents an estimate of the energy requirements for each upgrade option. Electricity costs are based on pumps, blowers, and mechanical operation, at the 2003 BC Hydro Business Rate structure. The natural gas expenditure for digester heating is estimated about 10% of total electricity bill (2002 record) at a rate of $11/GJ. Digester exhaust (50 to 60% methane under normal conditions) can be used for the boilers and co-generation engine operation in order to recovery energy.

TABLE 9.10
ENERGY REQUIREMENT OF EACH INTERIM UPGRADE OPTION

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Energy Requirement						
Electricity, kWh/yr	35,349,000	53,339,000	19,519,000	31,819,000	20,782,000	19,373,000
Natural Gas, GJ/yr	3,800	5,700	2,100	3,400	2,200	2,100
Electricity Cost	$2,079,000	$3,138,000	$1,148,000	$1,872,000	$1,222,000	$1,140,000
Natural Gas Cost	$42,000	$63,000	$23,000	$37,000	$24,000	$23,000
Total Energy Cost	$2,121,000	$3,200,000	$1,171,000	$1,909,000	$1,247,000	$1,162,000

9.1.11 Ability to Handle Load Variability

The maximum treatment flows of biological treatment are fixed at 250 ML/d (50% of ADWF) or 500 ML/d (100% of ADWF). Excess flow will bypass the secondary treatment and so under normal operating conditions the biological processes will not be overloaded. In most cases, CAS is able to handle load fluctuations more efficiently than the other options. RTF and HRAS, due to their high organic loading rate and short HRT respectively, are less tolerant of shock loading. The chemical dosing rate in a CEP process can be set to pace variations in the system load.

9.1.12 Visual Impact

The visual impact of the CAS and HRAS options will be no more adverse than that of the existing primary plant. The TF/SC option will have a similar visual impact as the Annacis Island and Lulu Island WWTPs. The CEP process will not be an issue from an aesthetic point of view as only the extra thickeners and digesters will be constructed. A green belt setback (vegetation or fence) can be considered in order to mitigate any visual impact upon adjacent public areas.

9.2 LIONS GATE

9.2.1 General

Roughing Trickling Filter (RTF), Biological Aerated Filter (BAF, and High-Rate Activated Sludge (HRAS) are the three biological processes that passed the first level of process screening and are considered for LGWWTP interim treatment upgrades. Chemical enhanced primary (CEP) followed by partial biological treatment with RTF, is also considered a potential scheme to reduce the capacity of subsequent biological treatment. The following five (5) upgrade options were developed, each with two different design capacities:

- Option 1: Primary + 50% average dry weather flow (ADWF) BAF (Series)
- Option 2A: 50% ADWF RTF + (Q – 50% ADWF) Primary (Parallel)
- Option 2B: 100% ADWF RTF + (Q – 100% ADWF) Primary (Parallel)
- Option 3: CEP + 50% ADWF RTF (Parallel)
- Option 4: 50% ADWF HRAS + (Q – 50% ADWF) Primary (Parallel).

Further analysis of these upgrade options is detailed in this section. Brief process descriptions, schematic flow diagrams, conceptual process designs and plant layouts, footprint requirements, sludge production, effluent quality projections, capital and operating and maintenance (O&M) cost estimates, process flexibility, environmental and social impacts and other factors are discussed in the following sections.

9.2.2 Description of Upgrade Options

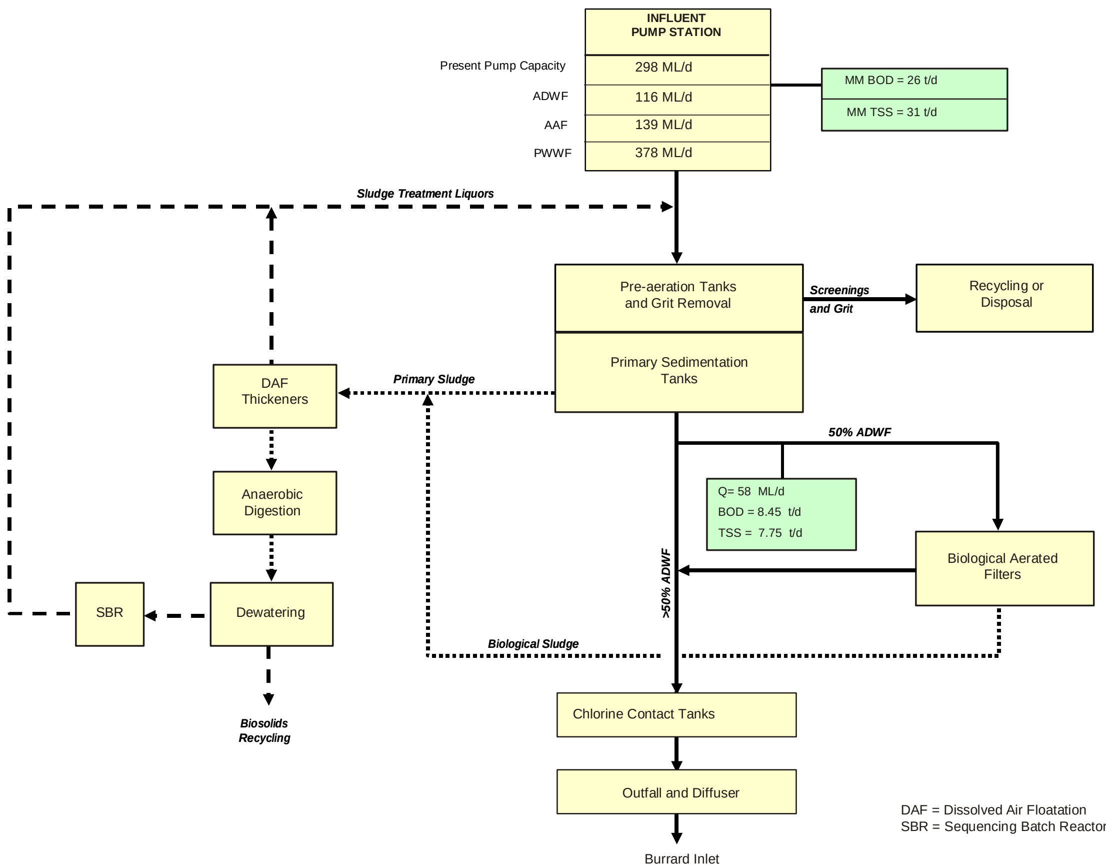
9.2.2.1 Option 1: Primary + 50% ADWF BAF

A process schematic of this option is illustrated in Figure 9.13. The preliminary (screen and grit removal) and primary (primary sedimentation tank) units are designed to treat the entire flow collected from the North Shore Sewage Area (NSSA). The BAF process is designed to provide 50% of average dry weather flow (ADWF) capacity (50% x 116 ML/d = 58 ML/d), at 50% of the maximum month flow (MMF) loading (8.45 t/d of BOD and 7.75 t/d of TSS) after the primary treatment units. The BAF process does not require final clarifiers to remove TSS and biosolids (biological sludge) generated in the biological process. After primary treatment flows greater than 50% of the ADWF will bypass the secondary treatment units and discharge directly to the chlorination system and the outfall.

The primary sludge and biological sludge are combined and thickened in dissolved air flotation (DAF) units. The thickened sludge is stabilized in the anaerobic digesters to achieve volatile solids reduction and pathogen reduction. The anaerobic digesters will continue to operate at thermophilic temperatures. The digested biosolids will continue to be dewatered and trucked to beneficial reuse. Centrate from the dewatering operations will be treated in a Sequencing

Batch Reactor (SBR) to convert the Ammonia to Nitrate and to reduce the high BOD, before the stream is returned to the inlet flow.

FIGURE 9.13
PROCESS SCHEMATIC OF LGWWTP INTERIM TREATMENT OPTION 1

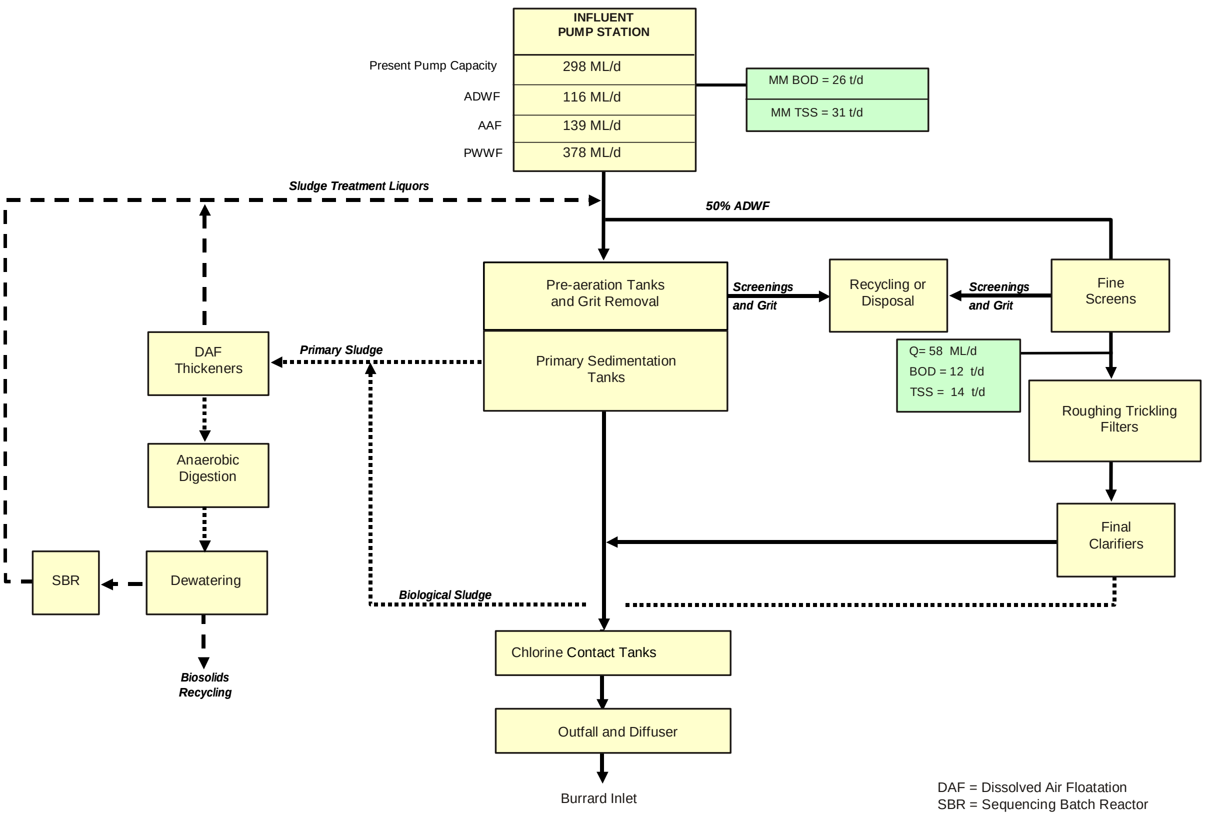


9.2.2.2 Option 2A: 50% ADWF RTF + Primary

A process schematic of this option is illustrated in Figure 9.14. A flow of 50% of ADWF is diverted from the flow to the Primary treatment system to the RTF in such a way that the amount of grit diverted is minimized. Fine screens protect the media of the RTF from being clogged by coarse material. The preliminary (screen and grit removal) and primary (primary sedimentation tank) units receive the balance of the flow collected from the North Shore Sewage Area (NSSA) and discharge directly to the chlorination system and the outfall. The RTF process is designed to provide 50% of average dry weather flow (ADWF) capacity (50% x 116 ML/d = 58 ML/d). This is associated with 50% of the maximum month flow (MMF) loading (12 t/d of BOD and 14 t/d of TSS). Final clarifiers remove TSS and biosolids (biological sludge) generated in the biological process. The flow then discharges to the chlorination system and the outfall.

The primary sludge and biological sludge are combined and thickened in dissolved air flotation (DAF) units. The thickened sludge is stabilized in the anaerobic digesters to achieve volatile solids reduction and pathogen reduction. The anaerobic digesters will continue to operate at thermophilic temperatures. The digested biosolids will continue to be dewatered and trucked to beneficial reuse. Centrate from the dewatering operations will be treated in a Sequencing Batch Reactor (SBR) to convert the Ammonia to Nitrate and to reduce the high BOD, before the stream is returned to the inlet flow.

FIGURE 9.14
PROCESS SCHEMATIC OF LGWWTP INTERIM TREATMENT OPTION 2A

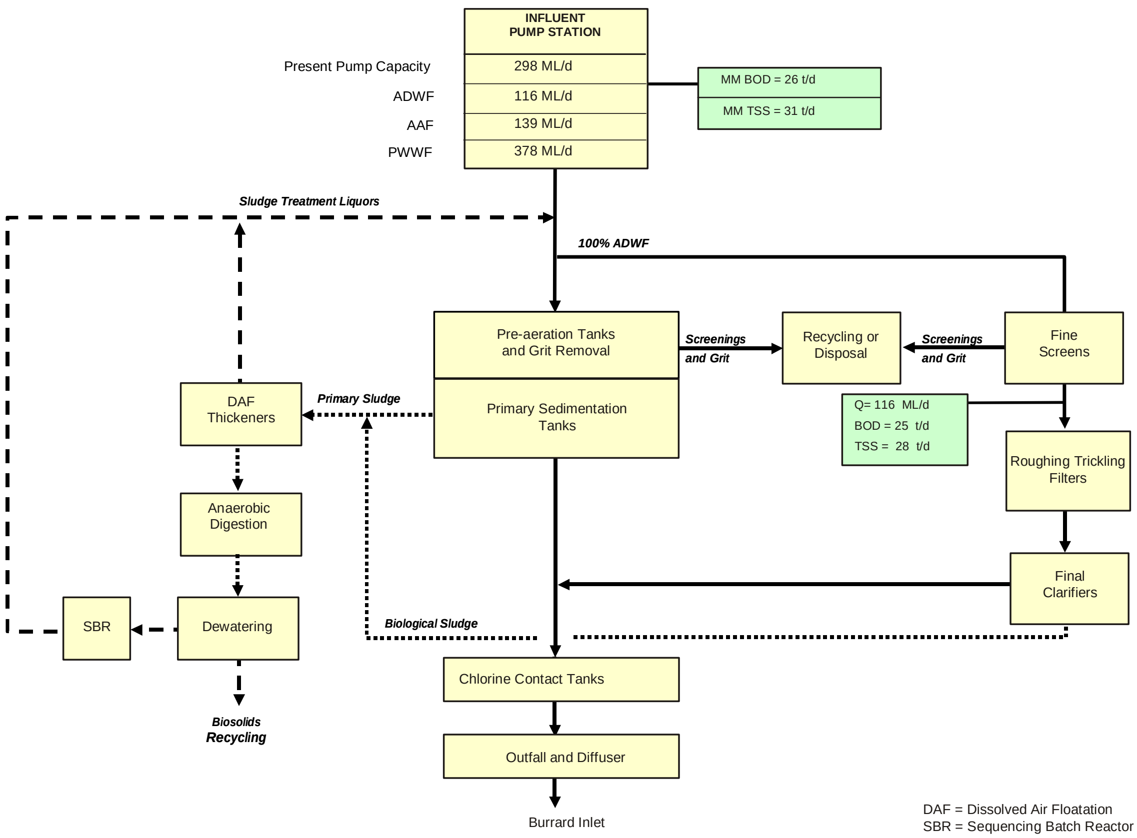


9.2.2.3 Option 2B: 100% ADWF RTF + Primary

This option is basically the same as option 2A except with different design flows and loads for the biological treatment process. The justification to increase the design flow for biological treatment from 50% to 100% of ADWF was based on the results of the small-scale testing. The objective of the testing was to determine treatment levels in order to improve LC_{50} test results. The results have indicated that a minimum of 50%~75% ADWF flow should receive biological treatment in order to achieve this goal. A process schematic is illustrated in Figure 9.15. The design flow and loads on the biological process are 100% of ADWF (116 ML/d) and 90% of MMF loadings after primary treatment (25 t/d of BOD and 28 t/d of TSS). The arrangements for solids handling and centrate treatment are the same as in Option 2A.

By treating 100% of ADWF, the footprint requirement, capital investment, O&M costs will be significantly higher than treating only 50% of ADWF. However, the effluent quality can be improved substantially, particularly during dry weather flow conditions.

FIGURE 9.15
PROCESS SCHEMATIC OF LGWWTP INTERIM TREATMENT OPTION 2B

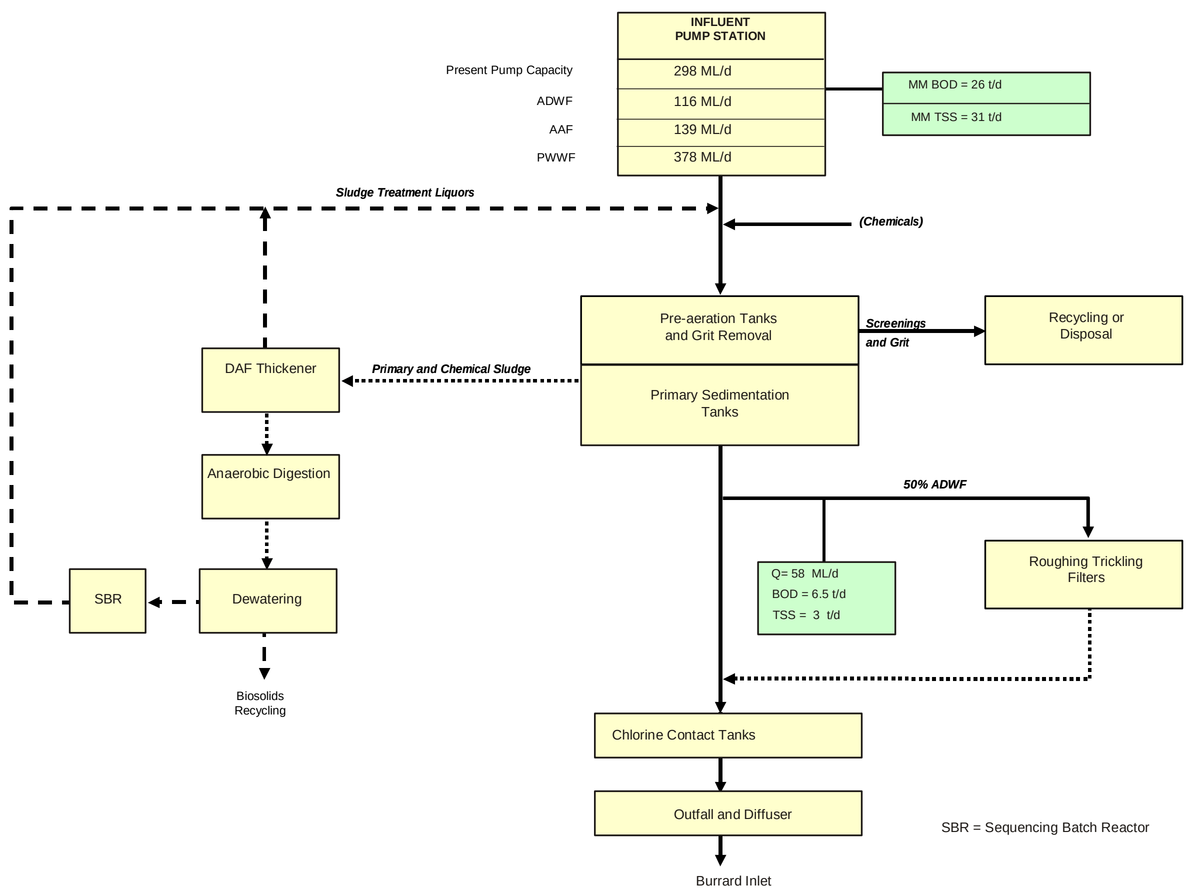


9.2.2.4 Option 3: CEP + 50% ADWF RTF

This option differs in a number of respects from Option 2A. Firstly primary treatment precedes the RTF, secondly the primary treatment is chemically enhanced and thirdly no final clarifiers are provided downstream of the RTF. Chemicals (alum and polymer) are applied prior to the primary sedimentation tanks to increase TSS and BOD removal efficiency by chemically enhanced precipitation. A process schematic is shown in Figure 9.16. The entire flow will receive preliminary and CEP treatment. Following the CEP process, 50% of the ADWF (50% x 116 ML/d = 58 ML/d) will be treated in the RTF process. The design loadings on the RTF process are 50% MMF loadings, i.e. 6.5 t/d of BOD and 3 t/d of TSS. The high level of TSS removal in the CEP allows space within the permit limits for discharge of biological solids from the RTF. The excess flow greater than 50% of ADWF will be treated by CEP and bypass the secondary units. This CEP effluent will combine with the secondary effluent to discharge to the chlorination system and the outfall.

The combined primary and chemical sludge will be collected in the primary sedimentation tanks and thickened in a DAF. The solids and centrate handling arrangements, subsequent to the thickeners, are the same as in Option 2A.

FIGURE 9.16
PROCESS SCHEMATIC OF LGWWTP INTERIM TREATMENT OPTION 3



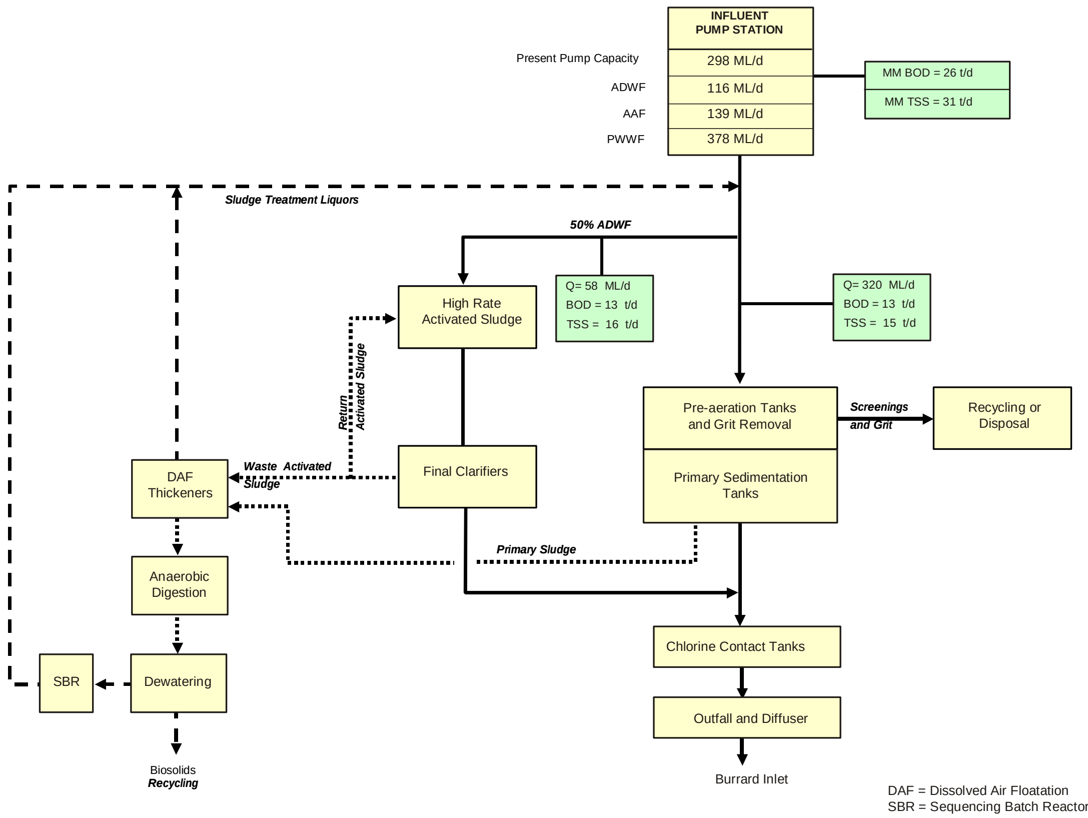
9.2.2.5 Option 4: 50% ADWF HRAS + (Q – 50% ADWF) Primary

HRAS will be used in conjunction with primary treatment in this option. Figure 9.17 shows a process schematic with the HRAS and primary treatment processes in parallel. Following preliminary screening a flow of 50% of ADWF is diverted from the flow to the Primary treatment system to the HRAS, in such a way that the amount of grit diverted is minimized. 50% of ADWF (50% x 116 ML/d = 58 ML/d) will be directed to the HRAS process for treatment. The remaining flow (greater than 58 ML/d) will be treated in the primary units only.

The primary and HRAS effluents will be combined and discharged to the chlorination system and the outfall.

TSS and biosolids from the HRAS units will be collected in the final clarifiers, combined with the primary sludge and thickened in the DAF units. The arrangement of solids handling and centrate treatment are the same as in Option 2A.

FIGURE 9.17
PROCESS SCHEMATIC OF LGWWTP INTERIM TREATMENT OPTION 4



9.2.3 Tank Size and Number of Units Required

A spreadsheet model was developed to carry out the conceptual process design and to determine the area of process units required for each upgrade option. The model summary is included in Appendix A. The unit process area and depth or height required, are listed in Table 9.11. Based on the area required to accommodate the process units for Build-out to Secondary (See Section 8.2 of Appendix 4) the HRAS process was eliminated, as it cannot be accommodated on the available site. Tank sizes have not been standardized across different options rather the area of tankage required has been shown. The modularization of the tankage would be carried out at the next stage of evaluation.

TABLE 9.11
LGWWTP UNIT PROCESS DIMENSIONS FOR EACH UPGRADE OPTION

YEAR			**2031 Interim**		
Option	Option 1 50% BAF	Option 2A 50% RTF	Option 2B 100% RTF	Option 3 CEP+RTF	Option 4 HRAS
Primary Clarifiers					
Existing Surface area (m^2)	2,742	2,742	2,742	2,742	2,742
Depth (m)	2.75	2.75	2.75	2.75	2.75
Total Area Required (m^2)	3,840	3,200	2,620	3,780	3,200
Additional Area Required (m^2)	1,098	458	0	1,038	458
BAF/Aeration/Solids Contact Tank					
Depth (m)	3.7				5
Total Area Required (m^2)	761				1,620
Roughing Trickling Filter (RTF)					
Depth (m)		6.3	5.6	2.9	
Total Area Required (m^2)		580	1160	580	
Trickling Filter (TF)					
Depth (m)					
Total Area Required (m^2)					
Final Clarifiers					
Depth (m)		4.5	4.5		4.5
Total Area Required (m^2)		806	1,611		1,933
DAF Thickeners					
Depth (m)	3.3	3.3	3.3	3.3	3.3
Total Area Required (m^2)	224.4	199.3	229.6	292.1	226.5
Digester					
Diameter (m) (No. 5)	21.5	20.5	20.5	26	19.8
Depth (m)	10.1	10.1	10.1	10.1	10
Unit Size (m^3)	3,667	3,334	3,334	5,362	3,079
Existing Volume (m^3) (Nos. 3 & 4)	6,220	6,220	6,220	6,220	6,220
Total Volume Required (m^3)	9,231	8,200	9,445	12,019	9,317
Additional Volume Required (m^3)	3,011	1,980	3,225	5,799	3,097
Centrifuge					
Digested Sludge Volume (m^3/d)	615	547	630	801	621
Existing Capacity (m^3/d) - 2 nos at 35 hrs/week	540	540	540	540	540
Additional Capacity Required (m^3/d)	75	7	90	261	81
Pressate Treatment (SBR)					
Depth (m)	4.5	4.5	4.5	4.5	4.5
Total Area Required (m^2/d)	227	201	232	299	227

9.2.4 Conceptual Site Layout

Conceptual site layouts for upgrade options 1, 2A, 2B and 3 are illustrated in Figures 9.18, 9.19, 9.20 and 9.21. No detailed layout was prepared for option 4 as there was insufficient space at the site. Expansion of the inlet pump station is shown adjacent to the existing pump station. Where required the primary sedimentation tanks (PST) are first expanded by the extension of the existing tanks 3 to 8 to the same length as tanks 1 and 2. Where required, separate additional tanks are provided. Large footprint units (bioreactors and clarifiers) are located to allow future expansion where possible. The smaller units (DAF, SBR and digesters) are located in the remaining available space along with the service buildings, such as pump stations and blower buildings. Where required, operations buildings will be expanded, by increasing the height of the existing buildings. The proposed use of UV disinfection for the secondary treated effluents in Build-out to Secondary, obviates the need for construction of additional chlorine contact tanks. The existing chlorine contact tanks would be retained for disinfection of primary treated effluents. Chemical treatment systems are accommodated on the site of the existing digesters No. 1 and 2, which are to be demolished.

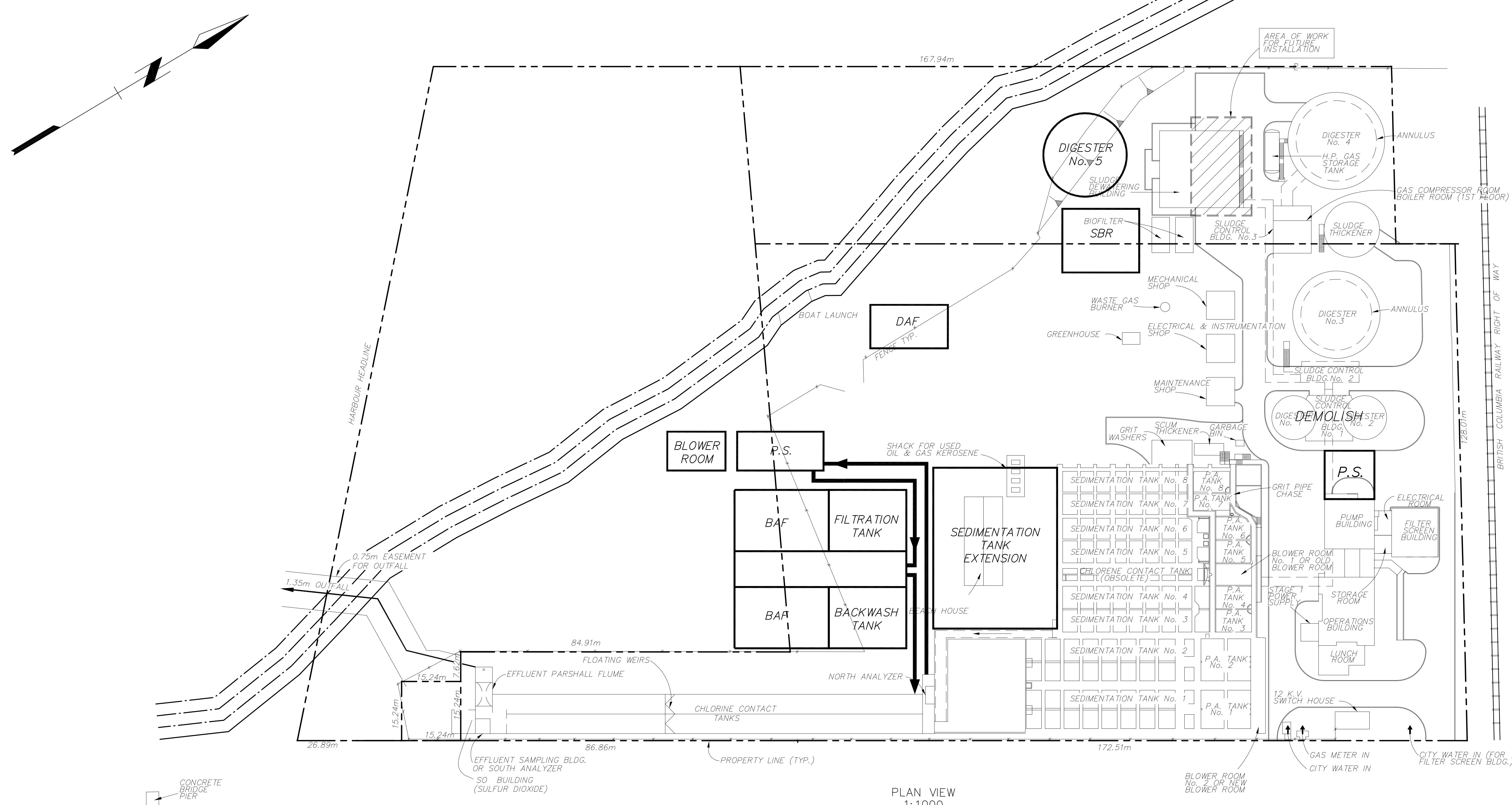
9.2.5 Projected Effluent Quality

The projected effluent quality (BOD and TSS concentrations) of each upgrade option are set out in Table 9.12, based on the annual average flow condition. Essentially, the biological process of Options 1, 2A, 3, and 4 will be operated at the design capacity most of the time, since the design AAF (139 ML/d) is much larger than 50% of ADWF (58 ML/d). With the same design treatment capacity, Option 1 (BAF process), Option 2A (RTF process) and Option 4 (HRAS process) will achieve the same effluent quality. Option 3 can be expected to achieve even better effluent quality due to the CEP. This is projected to be true even in the absence of final clarification, which adversely affects the TSS. Sloughing events could result in the TSS concentration exceeding the limits periodically. Option 2B with 100% of ADWF biological treatment capacity can deliver the highest TSS and BOD removals. Modelling of the inflow hydrograph and the associated BOD and TSS loads indicates that the mass of BOD or TSS treated approaches 90% as the plant capacity approaches 100% ADWF. The relationship for Summer conditions is illustrated in Figure 9.22.

TABLE 9.12
LGWWTP EFFLUENT CONCENTRATION PROJECTIONS OF EACH UPGRADE OPTION

	2002 Average	2031 Interim				
		Option 1	Option 2	Option 3	Option 4	Option 5
		50% BAF	50% RTF	100% RTF	CEP+RFT	HRAS
Design AAF (m^3/d)	87,760	139,000	139,000	139,000	139,000	139,000
BOD (mg/L)	91	73	73	35	48	73
SS (mg/L)	53	68	68	36	56	68

FIGURE 9.18



PLAN VIEW
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DWG. No. 415-1-103 ACAD DWG. OPTION2 1:1 03-12-05

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Issue	Date	Des'n	Dr'n	Chk'd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: P.T.L.
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

INTERIM TREATMENT–2031

OPTION 1: PRIMARY + 50% ADWF BAF

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

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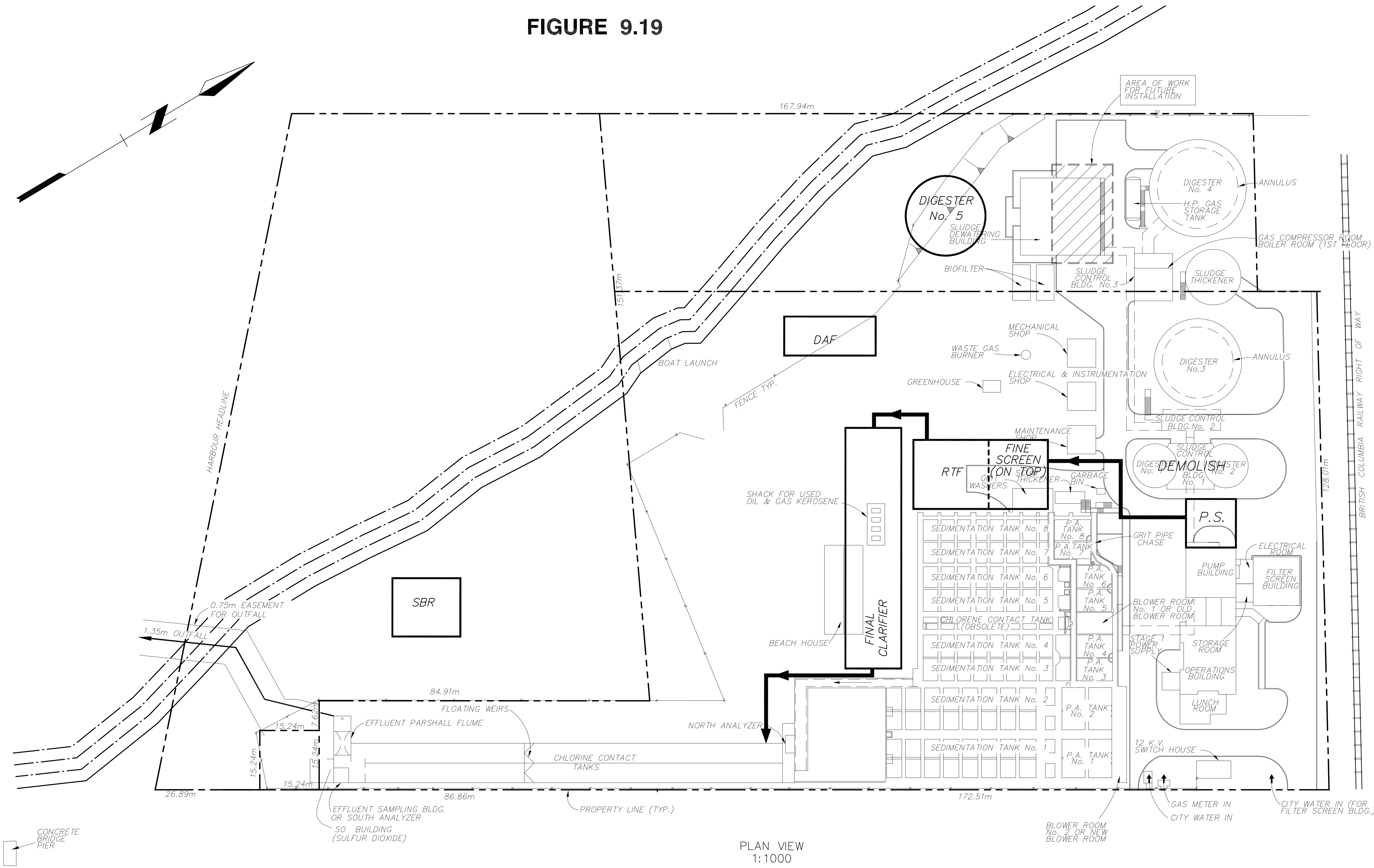
XREF DWG:

SAROS DWG. ID No.:

FIGURE 9.19



PLAN VIEW
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GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: PTL
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

INTERIM TREATMENT–2031

OPTION 2A: 50% ADWF RTF + PRIMARY

SCALE: AS SHOWN

DISTRICT FILE

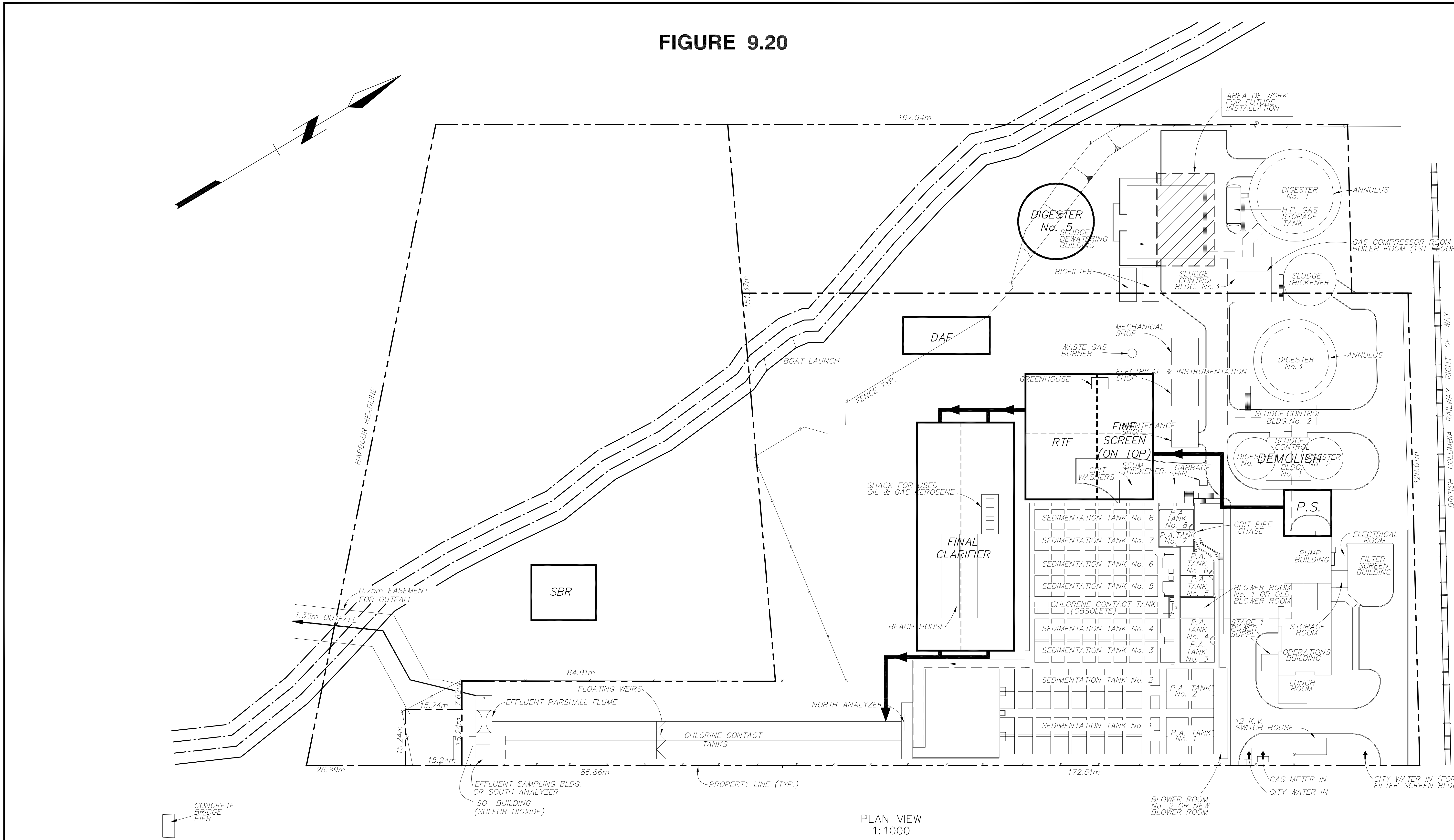
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:

FIGURE 9.20



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DWG. No. 415-1-103	ACAD DWG. OPTION5 1:1 03-12-05	Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description
0 — 20						Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

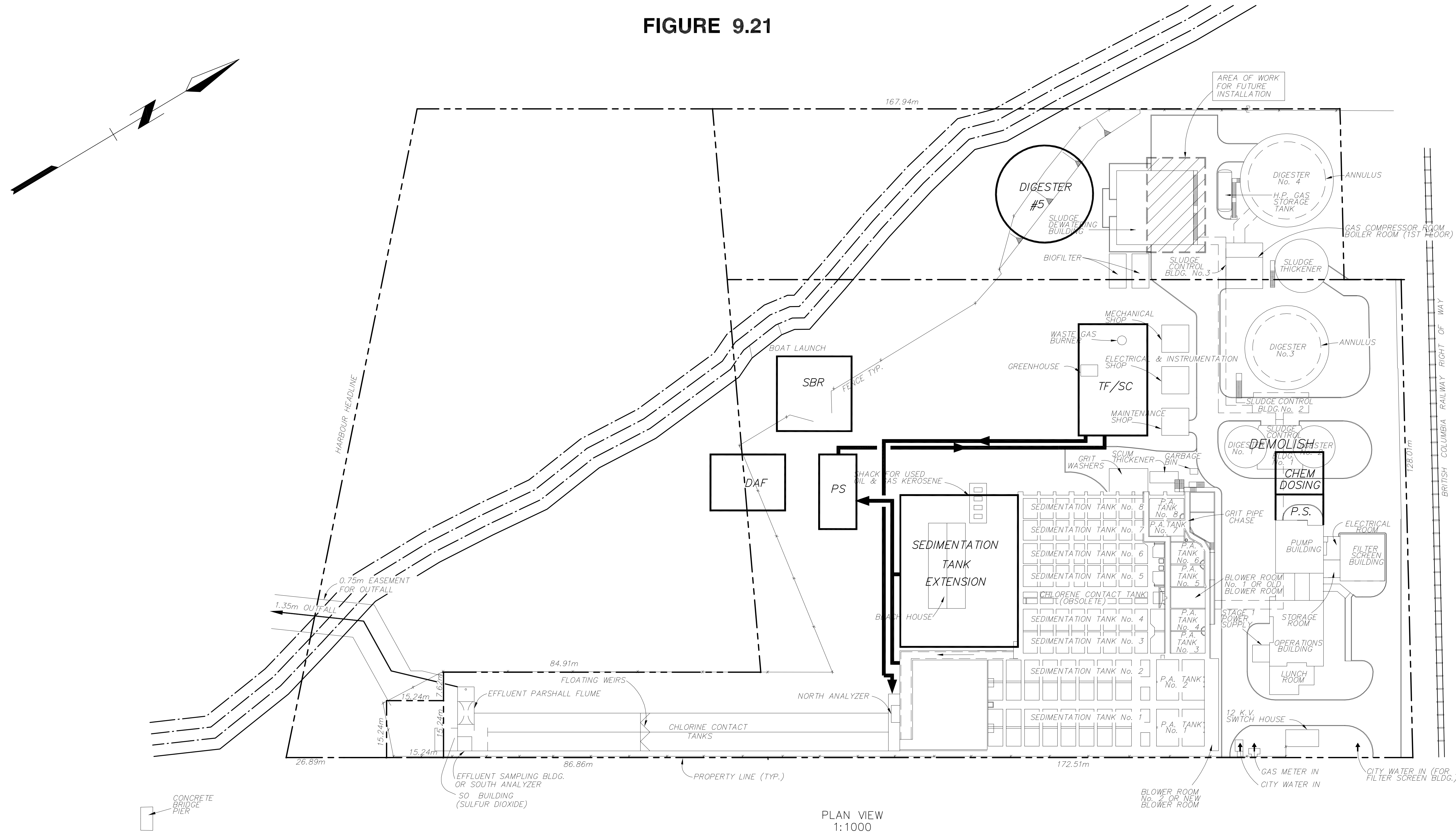
GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:	LIONS GATE WASTEWATER TREATMENT PLANT	SCALE: AS SHOWN
Drawn: PTL		
Checked:	INTERIM TREATMENT–2031	DISTRICT FILE
Approved	OPTION 2B: 100% ADWF RTF + PRIMARY	DOCUMENT CODE
Manager		

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO ➡

XREF DWG: SAROS DWG. ID No.:

FIGURE 9.21



PLAN VIEW
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DWG. No. 415-1-103 ACAD DWG. OPTION4 1:1 03-12-05

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GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: T.W.
Drawn: KS
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

INTERIM TREATMENT–2031

OPTION 3: CEP + 50% ADWF RTF

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

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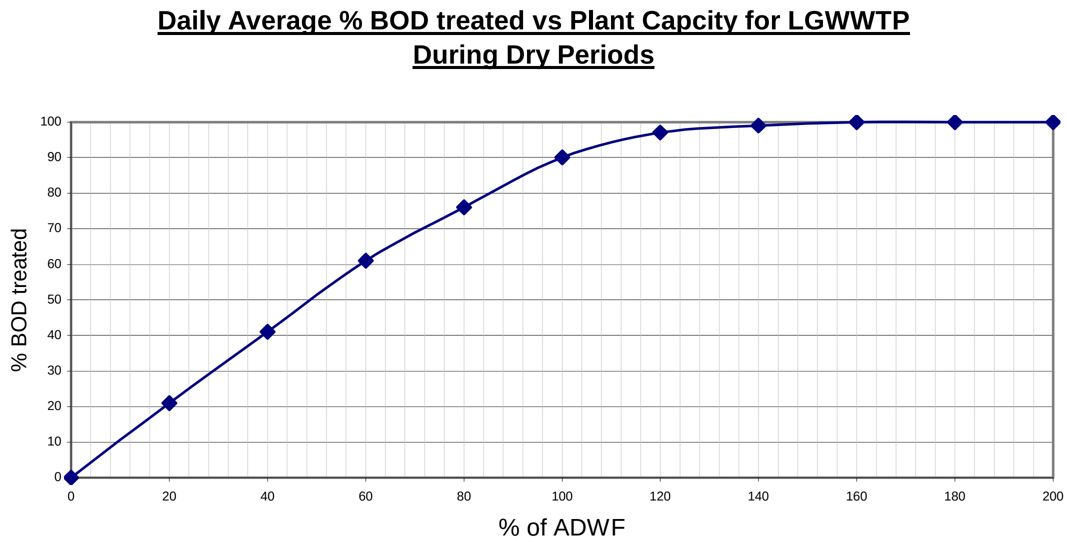
FIGURE 9.22
DAILY AVERAGE % BOD TREATED VS PLANT CAPACITY FOR LGWWTP



Improving the LC_{50} test results could be achieved by providing partial biological treatment. Since dry periods currently produce the poorest results the dry weather flow treatment capacity required to achieve an improvement must be assessed. Figure 9.22 illustrates the relationship between the mass of impurities treated (BOD) and the hydraulic treatment capacity taking into account the diurnal variation in flows during dry weather.

9.2.6 Sludge Production Projection

The projected primary, chemical and biological sludge production of each upgrade option is shown in Table 9.13. The estimated sludge volume at each sludge handling stage is shown in Table 9.13, including the, DAF float, digested sludge and dewatered sludge. The increased sludge production compared with the current level (2002 annual total) is summarized in Table 9.14 based on either dry mass or volume.

For the reasons noted below, Options 3 and 4 are expected to produce similar larger volumes of sludge:

- Option 3:The combination of CEP and RTF processes. Note that the introduction of a clarifier downstream of the RTF would increase the mass of sludge
- Option 4: Short sludge age of the process and the small mass of primary sludge.

Option 2A will produce the least sludge due to the longer effective sludge age and the better dewatering characteristics expected from digested trickling filter sludge as compared with activated sludge.

Option 2B will produce an equivalent amount of sludge based on the larger secondary treatment capacity.

Option 1 will produce more sludge than Option 2A because of the shorter effective sludge age.

It should to be noted that chemical sludge is less digestible in the stabilization process (e.g. anaerobic digestion), and less beneficial for reuse (e.g. agriculture and land application) because of lower available nutrient content.

TABLE 9.13
LGWWTP SLUDGE PRODUCTION FOR EACH INTERIM UPGRADE OPTION
(MAXIMUM MONTH)

YEAR		**2031 Interim**				
Option	Unit	**Option 1**	**Option 2A**	**Option 2B**	**Option 3**	**Option 4**
		50% BAF	**50% RTF**	**100% RTF**	**CEP+RTF**	**HRAS**
Raw Sludge/Biosolids						
Primary Sludge	T/d	16	8	2	0	8
CEP Sludge	T/d	0	0	0	28	0
Secondary Biosolids	T/d	6	11	20	0	14
Total Raw Sludge	T/d	22	19	22	28	22
Thickened Sludge						
Gravity Thickener (5%)	m^3/d	0	0	0	0	0
DAF (3.5%)	m^3/d	615	547	630	801	621
Total Thickened Sludge	m^3/d	615	547	630	801	621
Digested Sludge	m^3/d	615	547	630	801	621
Dewatered Sludge	m^3/d	49	43	50	53	55

TABLE 9.14
LGWWTP INCREASE OF SLUDGE COMPARED TO CURRENT LEVEL
(MAXIMUM MONTH)

YEAR	**2031 Interim**					
Option	Unit	Option 1	Option 2A	Option 2B	Option 3	Option 4
		50% BAF	50% RTF	100% RTF	CEP+RTF	HRAS
Raw Sludge	%	66	47	70	116	67
Thickened Sludge	%	137	110	142	208	139
Digested Sludge	%	137	110	142	208	139
Dewatered Sludge	%	117	92	122	135	143

9.2.7 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 9.15. The detailed breakdowns of the cost estimates are included in Appendix B.

Option 2A, which consists of 50% RTF in parallel with Primary treatment, has the lowest capital cost. It is followed by CEP with 50% RTF. The cost of this process is increased over Option 2A by the sludge handling costs. Option 2B, 100% RTF in parallel with Primary is the next highest cost. Option 1 50% BAF after primary treatment has the highest capital cost because of the cost of the bioreactor and, to a lesser extent, the additional primary sedimentation and sludge treatment.

TABLE 9.15
LGWWTP CAPITAL COSTS OF EACH INTERIM UPGRADE OPTION

YEAR	**2031 Interim**			
Option	Option 1 50% BAF	Option 2A 50% RTF	Option 2B 100% RTF	Option 3 CEP+50% RTF
CAPITAL COSTS				
Site Improvements	$4,057,000	$3,640,000	$3,842,000	$3,737,000
Chemical Dosing	$0	$0	$0	$500,000
Primary Clarifiers	$4,454,000	$1,858,000	$0	$4,210,000
Fine Screen	$0	$1,242,000	$2,484,000	$0
Trickling Filter	$0	$3,276,000	$5,897,000	$1,504,000
Bioreactor	$14,524,000	combined with RTF	combined with RTF	combined with RTF
Final Clarifiers	$0	$2,130,000	$4,260,000	$0
Gravity Thickeners	$0	$0	$0	$0
DAF Thickeners	$4,690,000	$4,166,000	$4,799,000	$6,107,000
Digesters	$3,100,000	$2,038,000	$3,320,000	$5,970,000
Mechanical Dewatering	$1,254,000	$1,254,000	$1,254,000	$1,254,000
SBR	$0	$0	$0	$0
UV	$0	$0	$0	$0
Odour Control System	$130,000	$440,000	$687,000	$239,000
Site Works	$1,018,000	$225,000	$254,000	$371,000
Admin/Maint. Building	$1,300,000	$1,300,000	$1,300,000	$1,300,000
Control System	$1,381,000	$863,000	$1,124,000	$1,008,000
Electrical Substation	$65,000	$55,000	$55,000	$55,000
Existing Facility Upgrades	$0	$0	$0	$0
Division 1 Cost	$798,000	$471,000	$636,000	$563,000
Engineering	$5,756,000	$3,598,000	$4,684,000	$4,201,000
Project Management/QA/QC	$1,439,000	$899,000	$1,171,000	$1,050,000
Contingency	$10,792,000	$6,746,000	$8,783,000	$7,877,000
Subtotal	$54,758,000	$34,202,000	$44,551,000	$39,946,000
Net GST, 0% of Sub-Total	$0	$0	$0	$0
Total Capital Costs	$54,758,000	$34,202,000	$44,551,000	$39,946,000

9.2.8 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs (2003 dollars) of each upgrade option at 2030 flows are shown in Table 9.16. The existing primary plant has a staff of 12. For all options it is estimated that the staff would increase to 15 persons. The chemical costs for Option 3 include alum and polymer.

Electricity costs are based on existing costs and installed power and energy use on the plant site and the current BC Hydro, Business, Medium Power Tariff. Natural gas consumption has been based on the mass of sludge to be digested and the present cost of gas.

Maintenance has been taken as the existing cost plus a fixed % per annum (2.35%) of the total improvement capital value.

Administration costs have been increased pro rata the annual average flow to the plant.

The residuals management costs are estimated based on a rate of $100/wet tonne for hauling, reuse (e.g. land application), and other fixed expenses, assuming that land application sites are available.

TABLE 9.16
LGWWTP OPERATING AND MAINTENANCE COSTS OF EACH INTERIM UPGRADE OPTION

YEAR	**2031 Interim**			
Option	Option 1	Option 2A	Option 2B	Option 3
	50% BAF	50% RTF	100% RTF	CEP+50% RTF
O&M COSTS				
Labour	$1,742,000	$1,742,000	$1,742,000	$1,742,000
Chemical Costs	$224,000	$207,000	$228,000	$1,916,000
Residuals Management	$1,782,000	$1,583,000	$1,823,000	$1,930,000
Energy	$528,000	$404,000	$467,000	$464,000
Repair/Maintenance	$1,768,000	$1,604,000	$1,686,000	$1,650,000
Administration and others	$773,000	$755,000	$764,000	$760,000
Land and building Lease	$332,000	$332,000	$332,000	$332,000
Total (O&M Costs)	$7,150,000	$6,627,000	$7,043,000	$8,794,000

The above costs include the 2002 operating and maintenance costs of $4,044,000.
Costs are in present (2003) values.

9.2.9 Life Cycle Cost Estimates

The preliminary life cycle cost (LCC) of each upgrade option is presented in Table 9.17 at present (2003) value, using a 6% per annum discount rate and a 27 year evaluation period (ending 2030). It has been assumed that the construction will commence in 2006 and be completed in 2007. Option 2A has the lowest LCC, followed by Options 2B, 3 and 1. The difference in capital cost between Options 3 and 1 has decreased significantly for the LCC because of the higher operating cost of CEP.

TABLE 9.17
LGWWTP LIFE CYCLE COST ESTIMATE OF EACH INTERIM UPGRADE OPTION

YEAR	**2031 Interim**			
Option	Option 1 50% BAF	Option 2A 50% RTF	Option 2B 100% RTF	Option 3 CEP+50% RTF
Total 27-yr. O&M Costs	$76,650,000	$72,130,000	$75,729,000	$90,874,000
Discounted Capital Costs	$44,674,000	$27,904,000	$36,348,000	$32,590,000
Total Capital and O & M Costs at present value	$121,325,000	$100,034,000	$112,077,000	$123,464,000

9.2.10 Flexibility of Phasing

Plant development can be facilitated by phasing in modular expansions from interim to build-out to secondary. In order to transition from RTF interim to TF/SC secondary treatment, it will be necessary to build the solids contact tanks at the interim stage. This is because, in order to reduce the footprint, the trickling filters must be constructed above the solids contact tanks. The aeration mixing system would not be included at this time. Increasing the height of the RTF units will allow them to be included as part of the build-out to secondary using the TF/SC process. Additional sludge handling units can be phased in when needed. All capital investment during the interim stage can be used in the build-out to secondary stage.

9.2.11 Energy Requirement

The major energy requirement for operating the secondary processes is in pumping and aeration. The existing influent pumps are still necessary to elevate the flow into the plant. Additional energy is required to raise the flow into the bioreactors (trickling filter or BAF or activated sludge) from where it flows by gravity to the final clarifier, and/or disinfection system and to the effluent outfall. Additional energy is required in the BAF and HRAS options for internal recycling (e.g. filter backwash, return activated sludge) and sludge handling operation (e.g. waste activated sludge, scum collection, DAF, anaerobic digesters, dewatering).

Aeration power is essential to the BAF, HRAS and SBR bioreactors. Ventilation and odour control is also required in the fine screening, RTF and anaerobic digester operation. Heat energy such as natural gas is needed to operate the digester at mesophilic or thermophilic condition. The energy requirements of each upgrade option are estimated in Table 9.18.

TABLE 9.18
LGWWTP ENERGY REQUIREMENT OF EACH INTERIM UPGRADE OPTION

YEAR	**2031 Interim**			
Option	Option 1	Option 2A	Option 2B	Option 3
	50% BAF	50% RTF	100% RTF	CEP+50% RTF
Energy Requirement				
Electricity, kWh/yr	7,860,000	5,620,000	6,600,000	6,060,000
Natural Gas, GJ/yr	12,300	11,300	12,500	14,700
Electricity Cost	$393,000	$281,000	$330,000	$303,000
Natural Gas Cost	$135,000	$124,000	$137,000	$162,000
Total Energy Cost	$528,000	$404,000	$467,000	$464,000

9.2.12 Ability to Handle Flow and Load Variability

The maximum flows to the biological treatment are fixed at 58 ML/d (50% of ADWF) or 116 ML/d (100% of ADWF). The excess flow will bypass secondary treatment, therefore these biological processes will not be hydraulically overloaded under normal operating conditions. The ability to increase the hydraulic load on the BAF is limited as it is operating near its design limit. The hydraulic load on the RTF can be increased significantly and will be limited by the pumping capacity. CEP can be set up to adjust the chemical dosing automatically in accordance with the flow variation.

The design loads on the biological process are based on the MML and will be highest when the concentrations of BOD and TSS in the feed are highest. This is expected to occur during dry spells. The ability to increase the organic load on the BAF is limited as it is operating near to its design limit. The organic load on the RTF can be significantly increased, at the cost of a reduction in SBOD removal efficiency. HRAS has less tolerance to shock loading due to its high organic loading rate and short HRT. CEP cannot be easily adjusted to meet variations in the load unless the necessary real-time monitoring equipment is installed.

9.2.13 Visual Impact

The HRAS option will have no more adverse visual impact than the existing primary plant. The RTF (TF/SC) options will have a similar visual impact as the Annacis Island and Lulu Island WWTPs. CEP processes have no aesthetic impact as only the extra thickeners and digesters will be constructed.

10 SECOND LEVEL OF SCREENING

The second level of screening was undertaken to assess the relative merits of the short list of processes based on the specific configurations. The detailed steps in the second level of screening included the following steps:

1. Establish the decision context

   The purpose of this study is not to provide a recommendation but to provide GVRD with a short list of technically viable options that are available for short term upgrades to meet either the permit requirements or to reduce effluent toxicity and options that are available for built-out to secondary.

2. Identify the options to be evaluated

   The first level of screening had identified options that would be analyzed in more detail for both interim upgrades and for build-out to secondary. The analysis of the five interim upgrade options for Iona Island and the four interim upgrade options is described in Sections 9.1 and 9.2 of this report (Appendix 3). The analysis of the three built-out to secondary options for Iona Island and the five built-out to secondary options for Lions Gate is described in Section 8.1 and 8.2 of Appendix 4.

   These options are summarized in Table 10.1.

TABLE 10.1
OPTIONS TO BE EVALUATED TO DEVELOP SHORT LIST

	Interim Upgrades	Build-out to Secondary
Iona Island	1. Primary treatment + 50% ADWF CAS 2. Primary treatment + 50% ADWF RTF 3. 50% ADWF HRAS and primary for balance 4. CEP + 50% ADWF RTF 5. CEP Only	1. Primary treatment + 100 % of 2 x ADWF TF/SC 2. Primary treatment + 100 % of 2 x ADWF CAS 3. CEP + 60% of 2 X ADWF CAS
Lions Gate	1. Primary + 50% ADWF BAF 2. 50% ADWF RTF or 100 % ADWF + Primary 3. CEP + 50% ADWF RTF 4. 50% ADWF HRAS + Primary	1. Primary + 100 % of 2 x ADWF TF/SC 2. Primary + 100 % of 2 x ADWF BAF 3. Primary + 100 % of 2 x ADWF HRAS 4. CEP + 60% of 2 X ADWF CAS 5. 100 % of 2 x ADWF TF/SC in parallel with primary

3. Identify criteria

   GVRD has requested that all evaluation criteria be grouped under three categories: cost, environmental and social. Since there are different biological, chemical and physical treatment processes being evaluated, the consulting team suggested that technical factors be added as a fourth category in the evaluation. The four categories and the factors in each grouping were discussed with the GVRD prior to starting scoring. The four categories and the criteria for evaluating the options in order to provide a short list of options are:

   Cost
   - Capital cost
   - Operating and maintenance cost
   - Life cycle cost was also added as a separate criteria because it was felt that it measured the impact of each option on the cash flow

   Technical
   - Ability to expand on site
   - Ability to handle load variation
   - Ease of phasing
   - Ability to upgrade for nitrogen removal
   - Resiliency of process
   - Compatibility of process with other GVRD plant

   Environmental
   - Energy use
   - Greenhouse gases
   - Sludge production
   - Effluent quality
   - Impact on wildlife habitat – applied to Iona Island only
   - Production of aerosols

   Social
   - Visual impact
   - Risk of odours
   - Traffic generation

4. Scoring

   The point values were filled in for each option as follows:

   - Cost – maximum points values for the lowest cost with the remainder being valued pro-rata the cost.
   -
   - Others – the best option receives maximum points values. The remaining options received a lower points value based on judgment.

These point values remain constant for Interim and Build-out to Secondary. Where the overall weighting of each category is changed, the weighting of each sub-category is changed pro-rata.

5. Weighting

GVRD requested that a triple bottom line approach including cost, environmental and social factors be considered in the development of the short list of options for interim upgrades and build-out to secondary. Because technical factors will have an impact on cost, it was agreed to combine technical factors with cost. GVRD also requested that a sensitivity analysis be carried out by three different weightings of the three categories of cost/technical, environmental and social. The different weightings by category are described in Table 10.2.

The weighting of the individual criteria within a category were assigned by dividing the weighting indicated in Table 10.2 by the number of criteria in each category. For example, in the case where environmental factors were assigned 20% of the weight and there are 5 criteria in this category, each criteria was assigned 4% of the weight.

The weighting against each category was adjusted to reflect the specifics of each site. For example wildlife habitat is not applicable to the Lions Gate plant and is weighted at 0%.

6. Combine weights and scores

Weights and score were combined. The tables with the combination of weights and score for the three different weighting systems are included in Appendix C of this report. The results of the weighted scores for the three different weighting systems are summarized in Tables 10.3 and 10.4 for Iona Island and in Tables 10.5 and 10.6 for Lions Gate.

TABLE 10.2
WEIGHTINGS BY CATEGORY
FOR SECOND LEVEL OF SCREENING

Category			
Weighting	Cost/Technical	Environmental	Social
1	50% (30% cost; 20% technical)	25%	25%
2	30% (24% cost; 6% technical)	50%	20%
3	30% (24% cost; 6% technical)	20%	50%

The variation in weighting allows the robustness of the assessment to be tested. The weightings in each category were further broken down using sub-divisions as shown on the example Table 10.2.

TABLE 10.3
IIWWTP BUILD-OUT TO SECONDARY TREATMENT
SUMMARY OF SECOND LEVEL OF SCREENING

	Option 1 – TF/SC		Option 2 – CAS		Option 3 – CEP + 60% CAS	
	Points	Rank	Points	Rank	Points	Rank
Cost & Technical @ 50%	89.6	1	85.0	2	84.7	3
Environmental @ 50%	91.6	1	82.8	3	82.9	2
Social @ 50%	85.5	2	88.4	1	84.2	3
Overall Rank	1		2		3	

TABLE 10.4
IIWWTP INTERIM TREATMENT
SUMMARY OF SECONDARY LEVEL OF SCREENING

	Option 1A		Option 1B		Option 2		Option 3		Option 4		Option 5	
	50% CAS		100% CAS		50% RTF		50% HRAS		CEP + 50% RTF		CEP	
	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking	Points	Ranking
Cost + Tech @ 50%	79.7	2	68.2	6	81.9	1	77.5	4	69.5	5	78.7	3
Environmental @ 50%	78.9	3	65.6	6	84.1	1	76.6	4	72.1	5	82.0	2
Social @ 50%	81.7	1	73.6	5	77.8	3	78.4	2	64.8	6	77.0	4
Overall Rank	2		5		1		4		6		3	

TABLE 10.5
LGWWTP BUILD-OUT TO SECONDARY TREATMENT
SUMMARY OF SECONDARY LEVEL OF SCREENING

	Option 1		Option 2		Option 4		Option 5	
	TF/SC		BAF		CEP + 60% TF/SC		Primary + TF/SC	
	Points	Rank	Points	Rank	Points	Rank	Points	Rank
Cost + Tech @ 50%	83.5	2	89.1	1	78.0	4	83.2	3
Environmental @ 50%	89.0	1	88.9	2	81.1	4	88.6	3
Social @ 50%	81.3	2	90.1	1	75.2	4	80.6	3
Rank	2		1		4		3	

TABLE 10.6
LGWWTP INTERIM TREATMENT
SUMMARY OF SECONDARY LEVEL OF SCREENING

	Option 1		Option 2		Option 4		Option 5	
	50% BAF		50% RTF		100% RTF		CEP + 50% RTF	
	Points	Rank	Points	Rank	Points	Rank	Points	Rank
Cost + Tech @ 50%	80.3	2	81.9	1	79.8	3	76.2	4
Environmental @ 50%	78.8	4	85.0	1	84.8	2	79.7	3
Social @ 50%	83.1	1	80.0	2	78.3	3	73.7	4
Rank	3		1		2		4	

7. Examine Results

The results of the evaluations were reviewed with GVRD at Workshop # 3 held on January 19, 2004. The results of the second level of screening for the build-out to secondary must be consider the ranking for the interim solution.

For Iona Island, TF/SC ranks first for the build-out to secondary when cost/technical and environmental factors are weighted at 50%. Similarly RTF for interim upgrades ranks first when cost/technical and environmental factors are weighted at 50%. TF/SC and RTF received a low score when social criteria are weighted at 50% because this project has a higher visual impact due to the high tanks and this process has a higher risk of odours. Adding more elaborate odour controls can mitigate odours and visual impacts can be mitigated in time by landscaping with trees. Considering that all other secondary plants in the GVRD have trickling filters, TF/SC for build-out to secondary and RTF for interim upgrades were short-listed. For the interim upgrades, trickling filters can be used as roughing trickling filters with or without CEP.

Regarding Lions Gate, biological aerated filters rank first for build-out to secondary when cost/technical and social criteria are weighted at 50%. When environmental factors are weighted at 50%, BAF and TF/SC are rated roughly the same. BAF is the process with the lowest footprint and the Lions Gate site is seriously constrained for space. BAF has roughly the same cost as TF/SC. The advantage of a small footprint is that it will be possible to expand the treatment on the current site without any land acquisition. For this reason, BAF was short-listed for the build-out to secondary. Because BAF was short-listed for the build-out to secondary, it is logical to use BAF with the interim upgrade options with or without CEP.

Since BAF was short listed for interim and build-out to secondary at Lions Gate, GVRD asked that BAF be also added to the short list of options for Iona Island.

8. Proposed Short List for Interim Upgrades and Build-out to Secondary

The rationale for the short list of options at Iona Island can be summarized as follows:

- The high ranking of trickling filter options for both Build-out to Secondary and Interim indicate a preference for this technology. This is reinforced by the following factors:
    - TF/SC is well known to the GVRD as Annacis Island, Lulu Island and NW Langley Plants all utilize the process.
    - TF/SC has a smaller footprint than the alternative CAS process allowing more flexibility in future expansion.
    - The lower capital cost of TF/SC is attractive.
    - The lower energy requirement of the process is attractive.

- The interim use of partial RTF can be easily incorporated into the TF/SC build-out.

- The use of CEP can be incorporated into partial RTF and be used only at times of high loads or can provide pre-treatment at times of high loads for TF/SC at build out.

- The favorable rating of BAF at Lions Gate and the cost being comparable to TF/SC indicates that BAF should be considered for installation on Iona Island also.

  From the above the options to be considered in the final evaluation are:

  <u>Build Out to Secondary Treatment</u>

  - TF/SC
  - BAF

  <u>Interim Treatment</u>

  - 50% RTF
  - CEP + 50% RTF (no secondary clarifier).
  - Together these allow the interpolation of any level of CEP.
  - In addition to the above, the option to upgrade existing processes to meet the existing effluent standards under the increasing loads should be assessed.

The rationale for the short list of options at Iona Island can be summarized as follows:

- The clear preference for BAF is reinforced by the fact that should one of the other options chosen; expansion of capacity beyond the 2046 projected flows and loads becomes problematic.

- The relatively poor ranking of processes incorporating full time CEP does not preclude the inclusion of CEP in the future provided it is utilized on a part time basis for addressing periods when loads are high.

From the above the options to be considered in the final evaluation are:

<u>Build-out to Secondary Treatment</u>

- BAF

<u>Interim Treatment</u>

- 50% BAF
- CEP + 50% BAF
- Together these allow the interpolation of any level of CEP.

The refinement of the short list of options for interim upgrades and build-out to secondary at Iona Island and at Lions Gate is covered in Appendix 10.

APPENDIX A: PROCESS DESIGN SUMMARY

IIWWTP INTERIM UPGRADE PROCESS DESIGN

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Average Dry Weather Flow (ML/d), ADWF	500	500	500	500	500	500
Average Annual Flow (ML/d), AAF	650	650	650	650	650	650
Peak Wet Weather Flow (ML/d), PWWF	1530	1530	1530	1530	1530	1530
Maximum Month BOD Loading (t/d), MM BOD	124	124	124	124	124	124
Maximum Month TSS Loading (t/d), MM TSS	113	113	113	113	113	113
Primary Sedimentation Tank						
Average Annual Flow (ML/d)	650	650	650	400	650	650
Peak Wet Weather Flow (ML/d)	1530	1530	1530	1280	1530	1530
Overflow Rate ($m^3/m^2/d$) - AAF	40	40	40	40	40	40
Overflow Rate ($m^3/m^2/d$) - PWWF	100	100	100	100	100	100
Surface Area (m^2) - AAF	16250	16250	16250	10000	16250	16250
Surface Area (m^2) - PWWF	15300	15300	15300	12800	15300	15300
Depth (m)	2.74	2.74	2.74	2.74	2.74	2.74
Volume (m^3) - AAF	44525	44525	44525	27400	44525	44525
Volume (m^3) - PWWF	41922	41922	41922	35072	41922	41922
Raw Influent BOD Loading (t/d)	124	124	124	62	124	124
Raw Influent TSS Loading (t/d)	113	113	113	56.5	113	113
Total Influent BOD Loading (t/d)	124	124	124	62	124	124
Total Influent TSS Loading (t/d)	113	113	113	56.5	113	113
Design PC BOD removal (%)	35%	35%	35%	35%	55%	55%
Design PC TSS removal (%)	50%	50%	50%	50%	80%	80%
PC Effluent BOD Loading (t/d)	81	81	81	40	56	56
PC Effluent TSS Loading (t/d)	57	57	57	28	23	23
PC Effluent BOD Conc. @ AAF (mg/L)	89	89	89	72	61	61
PC Effluent TSS Conc. @ AAF (mg/L)	72	72	72	59	29	29
Chemical Usage	N/A	N/A	N/A	N/A		
Alum Dosage (mg/L)					70	70
Polymer Dosage (mg/L)					0.5	0.5
Alum Volume - AAF (m^3/d)					69	69
Polymer Volume - AAF (m^3/d)					0.8	0.8
Alum Volume - PWWF (m^3/d)					163	163
Polymer Volume - PWWF(m^3/d)					1.9	1.9
Biological Treatment						N/A
Treating % of ADWF	50%	100%	50%	50%	50%	
Design Flow (ML/d)	250	500	250	250	250	
Treating % of MM BOD loading	50%	100%	50%	50%	50%	
Design BOD Loading (t/d)	40	81	40	62	28	
Aeration Basin			N/A		N/A	N/A
Design MLSS (mg/L) CAS or HRAS	2500	2500		1700		
MLVSS/MLSS	0.8	0.8		0.8		
Design F/M (kg BOD/kg MLVSS)	0.4	0.4		1.4		
Sludge Yield	0.75	0.75		1.10		
Solids Retention Time SRT (days)	4	4		2		
Aeration Basin Volume (m^3)	50375	100750		32563.025		
Surface Area Required (m^2)	10075	20150		6512.605		
Oxygen Requirement (kg O_2/kg BOD_5)	1.15	1.15		0.80		
Actual Oxygen Transfer Rate AOTR (t/d O_2)	46.3	92.7		49.6		
SOTR (t/d O_2)	102	204		109		
Air requirement (scfm)	30588	61175		32736		
Design Effluent BOD Concentration (mg/L)	20	20		30		

IIWWTP INTERIM UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
RTF	N/A	N/A		N/A		N/A
BOD Loading after Fine screen (t/d)			40		28	
TSS Loading after Fine screen (t/d)			57		11	
Design Trickling Filter Loading (kg BOD/m^3/d)			3.5		3.5	
Volume of Trickling Filter (m^3) - organic load			11514		7971	
Hydraulic Loading rate (m^3/m^2.d) Min = 45			100		100	
Area of Trickling Filter (m^2) by hydraulic loading			2500		2500	
Depth of Tower (m)			4.6		3.2	
Design sBOD removal (%)			61%		61%	
Sludge Age (days)			3.0		3.0	
Sludge Yield (kg TSS/kg BOD)			0.81		0.81	
sBOD (%) of TBOD			35%		35%	
Biodegradable TSS (%)			80%		80%	
Effluent BOD (t/d)			6		4	
Effluent SBOD (mg/L)			22		15	
Effluent TSS (t/d)			33		23	
Final Clarifier						N/A
Surface Overflow Rate (m^3/m^2/d) -Max flow	45	45	72	45	72	
Surface Area 1 (m^2) -SOR	5556	11111	3472	5556	3472	
Solids Loading Rate (kg/m^2/d)-Max Flow	150	150	150	150	150	
Surface Area 2 (m^2) -SLR	5208	10417	218	5667	151	
Depth (m)	4.5	4.5	4.5	4.5	4.5	
Volume (m^3)	25000	50000	15625	25500	15625	
HRT (hr) @ Design Flow	1.92	1.92	1.50	1.22	1.50	
Design Effluent TSS Concentration (mg/L)	20	20	20	30	20	
Thickener - Gravity (for PS)						
Raw Primary Sludge (t/d)	57	57	57	28	90	90
Chemical Sludge (t/d)					15	15
Total Primary/CEP Sludge (t/d)	57	57	57	28	106	106
Solids Concentration After Thickening (%)	5%	5%	5%	5%	5%	5%
Sludge Volume (m^3/d)	1130	1130	1130	565	2111	2111
Design Solids Loading (kg/m^2/d)	100	100	100	100	100	100
Surface Area (m^2)	565	565	565	283	1056	1056
Thickener - DAF (for WAS)						N/A
Waste Activated Sludge (t/d) WAS	26	53	28	60	18	
Solids Concentration After DAF (%)	4%	4%	4%	4%	4%	
Sludge Volume (m^3/d)	756	1513	790	1713	503	
Design Solids Loading (kg/m^2/d)	48	48	48	48	48	
Surface Area (m^2)	552	1103	576	1249	367	
Digester (Mesophilic Anaerobic)						
Digester VSS Loading (kg/d/m^3)	2.5	2.5	2.5	2.5	2.5	2.5
Sludge VSS/TSS Ratio	0.75	0.78	0.75	0.75	0.75	0.72
Digester Volume (m^3) by VSS loading	24893	34148	25243	26460	36950	30403
Un-digested dry tonnes (T/d)	83	109	84	88	123	106
Digested dry tonnes (T/d)	58	75	59	62	86	75
Design HRT (d)	20	20	20	20	20	20
Digested Sludge Solids (%)	3.1%	2.8%	3.1%	2.7%	3.3%	3.6%
VS destruction %	40%	40%	40%	40%	40%	40%
Digested sludge VSS (T/d)	37	51	38	40	55	46
Digested sludge VSS/TSS ratio	0.64	0.68	0.64	0.64	0.64	0.61
Digested Sludge Volume (m^3/d) (without dewatering)	1886	2643	1920	2278	2614	2111
Actual HRT (d)	13.19557	12.921016	13.148713	11.616181	14.134434	14.4
Digester Volume (m^3) by Design HRT	37729	52857	38396	45557	52283	42227

IIWWTP INTERIM UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2021 Interim**					
Option	Option 1A Primary + 50% ADWF CAS	Option 1B Primary + 100% ADWF CAS	Option 2 Primary + 50% ADWF RTF	Option 3 50% ADWF HRAS + (Q-50% ADWF) Primary	Option 4 CEP + 50% ADWF RTF	Option 5 CEP Only
Dewatering						
Centrifuge (L/min)	1200	1200	1200	1200	1200	1200
Days of Operation / week	5	5	5	5	5	5
Hours of operation / day	7	7	7	7	7	7
Sludge Cake (%) (dewatered)	27%	27%	30%	27%	35%	35%
Sludge Cake (m^3/d) (dewatered)	215	279	196	229	246	215
Estimated Effluent @ AAF						
BOD (mg/L)	62	36	63	56	44	61
SS (mg/L)	52	32	52	48	26	29

LGWWTP INTERIM UPGRADE PROCESS DESIGN

YEAR	**2031 Interim Summer Conditions**				
	Option 1	Option 2A	Option 2B	Option 3	Option 4
	50% BAF	50% RTF	100% RTF	CEP+50% RTF	50% HRAS
Average Dry Weather Flow (ML/d), ADWF	116	116	116	116	116
Average Annual Flow (ML/d), AAF	139	139	139	139	139
Peak Wet Weather Flow (ML/d), PWWF	378	378	378	378	378
Maximum Month BOD Loading (t/d), MM BOD	26	26	26	26	26
Maximum Month TSS Loading (t/d), MM TSS	31	31	31	31	31
Primary Clarifier					
Average Annual Flow (ML/d)	139	81	23	139	81
Peak Wet Weather Flow (ML/d)	378	320	262	378	320
Overflow Rate ($m^3/m^2/d$) - AAF	40	40	40	40	40
Overflow Rate ($m^3/m^2/d$) - PWWF	100	100	100	100	100
Surface Area (m^2) - AAF	3,625	2,025	575	3,475	2,025
Surface Area (m^2) - PWWF	3,840	3,200	2,620	3,780	3,200
Depth (m)	2.79	2.79	2.79	2.79	2.79
Volume (m^3) - AAF	10,114	5,650	1,604	9,695	5,650
Volume (m^3) - PWWF	10,714	8,928	7,310	10,546	8,928
Raw Influent BOD Loading (t/d)	26	13	3	26	13
Raw Influent TSS Loading (t/d)	31	16	3	31	16
Total Influent BOD Loading (t/d)	26	13	3	26	13
Total Influent TSS Loading (t/d)	31	16	3	31	16
PC Influent MM BOD Conc. @ AAF (mg/L)	187	160	113	187	160
PC Influent MM TSS Conc. @ AAF (mg/L)	223	191	135	223	191
Design PC BOD removal (%)	35%	35%	35%	55%	35%
Design PC TSS removal (%)	50%	50%	50%	80%	50%
PC Effluent BOD Loading (t/d)	17	8	2	12	8
PC Effluent TSS Loading (t/d)	16	8	2	6	8
PC Effluent BOD Conc. @ AAF (mg/L)	122	104	73	84	104
PC Effluent TSS Conc. @ AAF(mg/L)	112	96	67	45	96
Chemical Usage	**N/A**	**N/A**	**N/A**		**N/A**
Alum Dosage (mg/L)				70	
Polymer Dosage (mg/L)				0.5	
Alum($Al_2(SO_4)_3$) Volume - AAF (m^3/d)				14.8	
Polymer Volume - AAF (m^3/d)				0.2	
Alum($Al_2(SO_4)_3$) Volume - PWWF (m^3/d)				40.3	
Polymer Volume - PWWF(m^3/d)				0.5	
Biological Treatment					
Plant Capacity% of ADWF	50%	50%	100%	50%	50%
Design Flow (MLD)	58	58	116	58	58
Treating % of MM BOD / TSS loading	50%	50%	90%	50%	50%
Design BOD Loading (t/d)	8.5	13.0	23.4	5.9	13.0
Aeration Basin	**N/A**	**N/A**	**N/A**	**N/A**	
Design MLSS (mg/L)					2,000
MLVSS/MLSS					0.75
Design F/M (kg BOD/kg MLVSS)					1.07
Observed Sludge Yield					1.21
Solids Retention Time (days)					1
Aeration Basin Volume (m^3)					8,100
HRT (hr) @ Design Flow					3.4
Aeration Basin Depth (m)					5
Surface Area Required (m^2)					1,620
Return Activated Sludge % (RAS)					150%
Oxygen Requirement (kg O_2/kg BOD_5)					0.83
Actual Oxygen Transfer Rate AOTR (t/d)					9.3
SOTR (t/d O2)					20.6
Air requirement (sCFM)					6168
Design Effluent BOD Concentration (mg/L)					30

LGWWTP INTERIM UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2031 Interim Summer Conditions**				
	Option 1	Option 2A	Option 2B	Option 3	Option 4
	50% BAF	50% RTF	100% RTF	CEP+50% RTF	50% HRAS
BAF		**N/A**	**N/A**	**N/A**	**N/A**
Sludge Yield	0.9				
Sludge Age (days)	2				
Design Organic Loading (kg/m^3/d)	4.30				
Design Hydraulic Loading m^3/m^2/d-average	144				
Design Hydraulic Loading m^3/m^2/d-peak	144				
Backwash flow Ml/d	6				
Reactor Area (m^2) - organic load	531				
Reactor Area (m^2) - average flow	444				
Reactor Area (m^2) - peak flow	444				
Depth (m)	3.7				
Volume Required (m^3)	1965				
Oxygen Requirement (kg O_2/kg BOD_5)	2.00				
Actual Oxygen Transfer Rate AOTR (t/d)	13.4				
SOTR (t/d O2)	29.5				
Air requirement (sCFM)	8857				
Design Effluent BOD Concentration (mg/L)	30				
Design Effluent TSS Concentration (mg/L)	30				
TF/SC	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**
Design Trickling Filter Loading (kg BOD/m^3/d)					
Volume of Trickling Filter (m^3) - organic load					
Depth of Tower (m)					
Area of Trickling Filter (m^2)					
Design AAF Hydraulic Loading m^3/m^2/d-Minimum					
Average Hydraulic Loading rate (m^3/m^2.d)					
Design MLSS (mg/L)					
MLVSS/MLSS					
Design F/M (kg BOD/kg MLVSS)					
Observed Sludge Yield					
Effective "Solids Retention Time" (days)					
Aeration Basin Volume (m^3) sBOD Load					
HRT (hr) @ AAF					
Aeration Basin Depth (m)					
Foot Print Area Required (m^2) BOD					
Minimum HRT Requirement (hr)					
Aeration Basin Volume (m^3)- HRT					
Surface Area Required (m^2) - HRT					
Return Activated Sludge % (RAS)					
Oxygen Requirement (kg O_2/kg BOD_5)					
Actual Oxygen Transfer Rate AOTR (t/d)					
SOTR (t/d O_2)					
Air requirement (sCFM)					
Mixing requirement (m^3 air/m^3/min)					
Air requirement (sCFM)					
Oxygen Transfer Efficiency %					
Peak factor					
Oxygen Applied (kgO2/kgBOD applied)					
Air flow rate at 20oC and 1.0Atm (m3/min)					
Design Effluent BOD Concentration (mg/L)					

LGWWTP INTERIM UPGRADE PROCESS DESIGN (Cont'd)

YEAR	2031 Interim Summer Conditions				
	Option 1	Option 2A	Option 2B	Option 3	Option 4
	50% BAF	50% RTF	100% RTF	CEP+50% RTF	50% HRAS
RTF - Sidestream U/S of PST (via Finescreen)	**N/A**				**N/A**
Design Fine Screen BOD Removal (%)		2%	2%		
Design Fine Screen TSS Removal (%)		5%	5%		
BOD Loading after Fine screen (t/d)		13	23	6	
TSS Loading after Fine screen (t/d)		15	27	3	
Design Trickling Filter Loading (kg BOD/m^3/d)		3.5	3.5	3.5	
Volume of Trickling Filter (m^3) - organic load		3640	6552	1671	
Hydraulic Loading rate (m^3/m^2.d) Min = 45		100.0	100.0	100.0	
Area of Trickling Filter (m^2)		580	1160	580	
Depth of Tower (m)		6.3	5.6	2.9	
Design SBOD removal (%)		61%	61%	61%	
Sludge Age (days)		3	3	3	
Sludge Yield (kg TSS/kg BOD)		1.03	1.03	0.81	
sBOD (%)		35%	35%	35%	
Biodegradable TSS (%)		80%	80%	80%	
Oxygen Transfer Efficiency %		5%	5%	5%	
Peak factor		1	1	1	
Oxygen Applied (kgO2/kgBOD applied)		13.24	13.24	13.24	
Air flow rate at 20oC and 1.0Atm (m3/min)		419	755	193	
Effluent SBOD (t/d)		1.8	3.2	0.8	
Effluent SBOD mg/l		30.4	27.3	13.9	
Effluent TSS (t/d)		13	24	5	
Final Clarifier	**N/A**			**N/A**	
Surface Overflow Rate (m^3/m^2/d) -Max flow		72	72		45
Surface Area 1 (m^2) -SOR		806	1,611		1,289
Solids Loading Rate (kg/m^2/d)-Max Flow		-	-		150
Surface Area 2 (m^2) -SLR		-	-		1,933
Depth (m)		4.5	4.5		4.5
Volume (m^3) (with larger surface area)		3,625	7,250		8,700
HRT (hr) @ Design Flow + RAS		1.50	1.50		1.44
Design Effluent TSS Concentration (mg/L)		30	30		30
Thickener - Gravity (for PS) insert Y or N	N	N	N	N	N
Raw Primary Sludge (t/d)					
Chemical Sludge (t/d)					
Total Primary/CEP Sludge (t/d)					
Solids Concentration After Thickening (%)	5%	5%	5%	5%	5%
Sludge Volume (m^3/d)					
Design Solids Loading MML (kg/m^2/d)	100	100	100	100	100
Surface Area (m^2)					
Thickener - DAF (for WAS or Combined Primary)	**Co-DAF**	Co-DAF	Co-DAF	Co-DAF	Co-DAF
Sludge (t/d)	21.5	19.1	22.0	28.0	21.7
Solids Concentration After DAF (%)	3.5%	3.5%	3.5%	3.5%	3.5%
Sludge Volume (m^3/d)	615	547	630	801	621
Design Solids Loading (kg/m^2/d) 48 for WAS 96 for Co-DAF	96.0	96.0	96.0	96.0	96.0
Surface Area (m^2)	224	199	230	292	226
Digester					
Digester VSS Loading (kg/d/m^3)	2.2	2.2	2.2	2.2	2.2
Sludge VSS/TSS Ratio	80%	80%	80%	85%	80%
Digester Volume (m^3) by VSS loading	7,832	6,957	8,014	10,835	7,905
Un-digested dry tonne (T/d)	22	19	22	28	22
Digested dry tonne (T/d)	15	13	15	19	15
Design HRT (d)	15	15	15	15	15
Digested Sludge Solids (%)	2.4%	2.4%	2.4%	2.3%	2.4%
VS destruction %	40%	40%	40%	40%	40%
Digested Sludge Volume (m^3/d)	615	547	630	801	621
Actual HRT (d)	13	13	13	14	13
Digester Volume (m^3) by Design HRT	9,231	8,200	9,445	12,019	9,317
Required Volume (Max) m^3	9,231	8,200	9,445	12,019	9,317

LGWWTP INTERIM UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2031 Interim Summer Conditions**				
	Option 1	Option 2A	Option 2B	Option 3	Option 4
	50% BAF	50% RTF	100% RTF	CEP+50% RTF	50% HRAS
Dewatering					
Centrifuge (L/min)	900	900	900	900	900
Days of Operation / week	5	5	5	5	5
Hours of operation / day	7	7	7	7	7
No. Centrifuges	2.3	2.0	2.3	3.0	2.3
Sludge Cake (%) (dewatered)	30%	30%	30%	35%	27%
Sludge Cake (m3/d) (dewatered)	49	43	50	53	55
Pressate Treatment SBR					
Pressate volume (m3/d)	567	503	580	748	566
SBR Volume (m3/d) = 1.8 x Pressate vol.	1,020	906	1,043	1,347	1,020
Depth of Reactor (m)	4.5	4.5	4.5	4.5	4.5
Area of Reactor (m2/d)	227	201	232	299	227
Effluent Standard					
BOD mg/l	130	130	130	130	130
TSS mg/l	130	130	130	130	130
Estimated Effluent @ Max Flow					
BOD (mg/L)	27	27	13	18	27
SS (mg/L)	25	25	13	21	25
Estimated Effluent @ AAF/ 2*ADWF					
BOD (mg/L)	73	73	35	48	73
SS (mg/L)	68	68	36	56	68

APPENDIX B: CAPITAL COST ESTIMATES

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 1A, PRIMARY + 50% ADWF CAS

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			215000	m^3	$10	$2,150,000
Preloading (1.5 m & 4 m)			88000	m^3	$15	$1,320,000
Ground densification - new			600000	m^3	$8	$4,800,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$925,000
Total for Site Improvement						$17,195,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Aeration Basin	4	14190	56760	m^3	$360	$20,433,600
Secondary Clarifiers	4	1520	6080	m^2	$2,140	$13,011,200
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	1.8	314	551.5625	m^2	$21,200	$11,693,125
Digesters	2.1	8520	17911.09	m^3	$940	$16,836,426
Mechanical Dewatering				l.s.		$0
Site Works:						
Pumping to Bioreactor			360	HP	$3,750	$1,350,000
Roads/grading				l.s.		$250,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			910	m	$450	$409,500
2400 mm effluent			225	m	$1,925	$433,125
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$2,478,974
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$85,940,950**
Division 1 Cost		2.5%				$1,718,649
Engineering		16%				$13,750,552
Project Management/ Quality Control		4%				$3,437,638
Contingency		30%				$25,782,285
Sub-total						**$130,630,074**
Net GST (0%)						$0
Total (Capital Costs)						**$130,630,074**

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 1B, PRIMARY + 100% ADWF CAS

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			360000	m^3	$10	$3,600,000
Preloading (1.5 m & 4 m)			130000	m^3	$15	$1,950,000
Ground densification - new			1000000	m^3	$8	$8,000,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$1,850,000
Total for Site Improvement						$23,400,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Aeration Basin	8	14190	113520	m^3	$360	$40,867,200
Secondary Clarifiers	8	1520	12160	m^2	$2,140	$26,022,400
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	3.5	314	1103.125	m^2	$21,200	$23,386,250
Digesters	3.9	8520	33038.41	m^3	$940	$31,056,108
Mechanical Dewatering				l.s.		$0
Site Works:						
Pumping to Bioreactor			720	HP	$3,750	$2,700,000
Roads/grading				l.s.		$500,000
750 mm RAS			1400	m	$500	$700,000
600 mm WAS			1920	m	$450	$864,000
2400 mm effluent			450	m	$1,925	$866,250
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$4,853,278
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$156,715,486**
Division 1 Cost		2.5%				$3,332,887
Engineering		16%				$25,074,478
Project Management/ Quality Control		4%				$6,268,619
Contingency		30%				$47,014,646
Sub-total						**$238,406,117**
Net GST (0%)						$0
Total (Capital Costs)						**$238,406,117**

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 2, PRIMARY + 50% ADWF RTF

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			215000	m^3	$10	$2,150,000
Preloading (1.5 m & 4 m)			88000	m^3	$15	$1,320,000
Ground densification - new			600000	m^3	$8	$4,800,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$925,000
Total for Site Improvement						$17,195,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Roughing Trickling Filter	2	5586	11172	m^3	$1,040	$11,618,880
Solids Contact	0	14190	0	m^3	$360	$0
Secondary Clarifiers	3	1520	4560	m^2	$2,140	$9,758,400
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	1.8	314	565.2	m^2	$21,200	$11,982,240
Digesters	2.2	8520	18744	m^3	$940	$17,619,360
Mechanical Dewatering				l.s.		$0
Site Works:						
Pumping to Bioreactor			360	HP	$3,000	$1,080,000
Roads/grading				l.s.		$250,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			910	m	$450	$409,500
2400 mm effluent			225	m	$1,925	$433,125
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		7%		l.s.		$4,500,000
Electrical substation (allow)				l.s.		$1,500,000
Odour Control				l.s.		$1,000,000
Existing Facility Upgrades						$0
Sub-Total						**$77,696,505**
Division 1 Cost		2.5%				$1,512,538
Engineering		16%				$12,431,441
Project Management/ Quality Control		4%				$3,107,860
Contingency		30%				$23,308,952
Sub-total						**$118,057,295**
Net GST (0%)						$0
Total (Capital Costs)						**$118,057,295**

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 3, 50% ADWF HRAS + (Q-50% ADWF) PRIMARY

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			250000	m^3	$10	$2,500,000
Preloading (1.5 m & 4 m)			97000	m^3	$15	$1,455,000
Ground densification - new			685000	m^3	$8	$5,480,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$925,000
Total for Site Improvement						$18,360,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Aeration Basin	3	14190	42570	m^3	$360	$15,325,200
Secondary Clarifiers	4	1520	6080	m^2	$2,140	$13,011,200
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	4	314	1256	m^2	$21,200	$26,627,200
Digesters	3	8520	25560	m^3	$940	$24,026,400
Mechanical Dewatering				l.s.		$0
Site Works:						
Pumping to Bioreactor			360	HP	$3,750	$1,350,000
Roads/grading				l.s.		$250,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			910	m	$450	$409,500
2400 mm effluent			225	m	$1,925	$433,125
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$3,159,600
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$104,802,225**
Division 1 Cost		2.5%				$2,161,056
Engineering		16%				$16,768,356
Project Management/ Quality Control		4%				$4,192,089
Contingency		30%				$31,440,668
Sub-total						**$159,364,393**
Net GST (0%)						$0
Total (Capital Costs)						**$159,364,393**

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 4, CEP + 50% ADWF RTF

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			215000	m^3	$10	$2,150,000
Preloading (1.5 m & 4 m)			88000	m^3	$15	$1,320,000
Ground densification - new			600000	m^3	$8	$4,800,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$925,000
Total for Site Improvement						$17,195,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Trickling Filters	2	4875	9750	m3	$1,065	$8,288,000
Solids Contact	0	14190	0	m^3	$360	$0
Secondary Clarifiers	3	1520	4560	m^2	$2,140	$9,758,400
Gravity Thickeners	2	308	616	m^2	$4,500	$2,772,000
DAF Thickeners	1.2	314	376.8	m^2	$21,200	$7,988,160
Digesters	3.8	8520	32376	m^3	$940	$30,433,440
Mechanical Dewatering				l.s.		$0
Chemical feed system						$1,500,000
Site Works:						
Pumping to Bioreactor			360	HP	$3,000	$1,080,000
Roads/grading				l.s.		$250,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			910	m	$450	$409,500
2400 mm effluent			225	m	$1,925	$433,125
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$2,369,600
Electrical substation (allow)				l.s.		$1,500,000
Odour Control				l.s.		$1,000,000
Existing Facility Upgrades						$0
Sub-Total						**$85,327,225**
Division 1 Cost		2.5%				$1,703,306
Engineering		16%				$13,652,356
Project Management/ Quality Control		4%				$3,413,089
Contingency		30%				$25,598,168
Sub-total						**$129,694,143**
Net GST (0%)						$0
Total (Capital Costs)						**$129,694,143**

IIWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 5, CEP ONLY

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			70000	m^3	$10	$700,000
Preloading (1.5 m & 4 m)			40000	m^3	$15	$600,000
Ground densification - new			200000	m^3	$8	$1,600,000
Ground densification - exist			0	m^3	$8	$0
Soil anchors - existing			0	each	$4,000	$0
Dewatering				l.s.		$370,000
Total for Site Improvement						$11,270,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Aeration Basin	0	14190	0	m^3	$360	$0
Secondary Clarifiers	0	1520	0	m^2	$2,140	$0
Gravity Thickeners	2	308	616	m^2	$4,500	$2,772,000
DAF Thickeners	0	314	0	m^2	$21,200	$0
Digesters	2.6	8520	22152	m^3	$940	$20,822,880
Mechanical Dewatering				l.s.		$0
Chemical feed system						$1,500,000
Site Works:						
Pumping to Bioreactor			0	HP	$3,750	$0
Roads/grading				l.s.		$250,000
750 mm RAS			0	m	$500	$0
600 mm WAS			0	m	$450	$0
2400 mm effluent			450	m	$1,925	$866,250
Admin/Maint Building	0	5000	0	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$943,795
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$39,924,925**
Division 1 Cost		2.5%				$716,373
Engineering		16%				$6,387,988
Project Management/ Quality Control		4%				$1,596,997
Contingency		30%				$11,977,478
Sub-total						**$60,603,761**
Net GST (0%)						$0
Total (Capital Costs)						**$60,603,761**

LGWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 1, PRIMARY + 50% ADWF BAF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)			1	l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	5364	14	75,096	m^3	$8	$600,768
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			2,960	m^2	$100	$296,000
Total for Site Improvement						$4,056,768
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	5.9	186	1,098	m^2	$4,056	$4,453,619
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
BAF			1	l.s.	$14,524,084	$14,524,084
Secondary Clarifiers	0.0	908	0	m^2	$2,644	$0
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.0	113	224	m^2	$20,905	$4,690,255
Digesters	0.8	3667	3,011	m^3	$1,030	$3,099,982
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR				m^3		$0
UV	-			ML/d PWWF		$0
Odour Control	Allowance					$130,000
Site Works:						
Pumping to Bioreactor			75	kW	$9,879	$743,514
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1050mm dia.)		1050	166.2	m	$1,050	$174,510
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			34,527,009	%	4.00%	$1,381,080
Electrical substation		845	1	l.s.	$65,000	$65,000
Existing Facility Upgrades						$0
Sub-Total						**$35,973,090**
Division 1 Cost		2.5%				$797,908
Engineering		16%				$5,755,694
Project Management/ Quality Control		4%				$1,438,924
Contingency		30%				$10,791,927
Sub-total						**$54,757,543**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$54,757,543**

LGWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 2A, PRIMARY + 50% ADWF RTF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	2947.32	14	41,262	m^3	$8	$330,100
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			1,500	m^2	$100	$150,000
Total for Site Improvement						$3,640,100
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	2.5	186	458	m^2	$4,056	$1,857,703
Fine Screens	-	-	58	ML/d	$21,413	$1,241,964
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	1.8	1972	3,640	m^3	$900	$3,276,000
Bioreactor	0.0	0	0	m^2		combined with RTF
Secondary Clarifiers	1.0	804	806	m^2	$2,644	$2,130,238
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	1.8	113	199	m^2	$20,905	$4,166,159
Digesters	0.6	3334	1,980	m^3	$1,030	$2,038,014
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR				m^3		$0
UV	-			ML/d PWWF		$0
Odour Control	Allowance					$440,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1050mm dia.)		1050	119	m	$1,050	$124,950
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			21,569,405	%	4.00%	$862,776
Electrical substation		503	1	l.s.	$55,000	$55,000
Existing Facility Upgrades						$0
Sub-Total						**$22,487,181**
Division 1 Cost		2.5%				$471,177
Engineering		16%				$3,597,949
Project Management/ Quality Control		4%				$899,487
Contingency		30%				$6,746,154
Sub-total						**$34,201,949**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$34,201,949**

LGWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 2B, 100% ADWF RTF + PRIMARY

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	3682.32	14	51,552	m^3	$8	$412,420
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			2,700	m^2	$100	$270,000
Total for Site Improvement						$3,842,420
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	0.0	186	0	m^2	$4,056	$0
Fine Screens	-	-	116	ML/d	$21,413	$2,483,929
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	3.7	1774	6,552	m^3	$900	$5,896,800
Bioreactor	0.0	0	0	m^2		combined with RTF
Secondary Clarifiers	1.8	908	1,611	m^2	$2,644	$4,260,477
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.0	113	230	m^2	$20,905	$4,798,907
Digesters	1.0	3334	3,225	m^3	$1,030	$3,320,141
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR			0	m^3		$0
UV	-			ML/d PWWF		$0
Odour Control	Allowance					$687,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1200mm dia.)		1200	128.5	m	$1,200	$154,200
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			28,098,151	%	4.00%	$1,123,926
Electrical substation		689	1	l.s.	$55,000	$55,000
Existing Facility Upgrades						$0
Sub-Total						**$29,277,077**
Division 1 Cost		2.5%				$635,866
Engineering		16%				$4,684,332
Project Management/ Quality Control		4%				$1,171,083
Contingency		30%				$8,783,123
Sub-total						**$44,551,481**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$44,551,481**

LGWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 3, CEP + 50% ADWF RTF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	4077.12	14	57,080	m^3	$8	$456,637
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			1,200	m^2	$100	$120,000
Total for Site Improvement						$3,736,637
Treatment Components:						
Chemical Dosing	Allowance					$500,000
Primary Clarifiers	5.6	186	1,038	m^2	$4,056	$4,210,252
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	1.8	905	1,671	m^3	$900	$1,504,286
Bioreactor	0.0	0	0	m^2		combined with RTF
Secondary Clarifiers	0.0	0	0	m^2	$2,644	$0
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.6	113	292	m^2	$20,905	$6,106,616
Digesters	1.1	5362	5,799	m^3	$1,030	$5,969,933
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR			0	m^3		$0
UV	-			ML/d PWWF		$0
Odour Control	Allowance					$239,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1050 mm dia.)		1050	258.5	m	$1,050	$271,425
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			25,192,425	%	4.00%	$1,007,697
Electrical substation		638	1	l.s.	$55,000	$55,000
Existing Facility Upgrades						$0
Sub-Total						**$26,255,122**
Division 1 Cost		2.5%				$562,962
Engineering		16%				$4,200,820
Project Management/ Quality Control		4%				$1,050,205
Contingency		30%				$7,876,537
Sub-total						**$39,945,645**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$39,945,645**

LGWWTP CAPITAL COST ESTIMATE: INTERIM OPTION 4, 50% ADWF HRAS + PRIMARY

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	6298.38	14	88,177	m^3	$8	$705,419
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering				m^2	$100	$0
Total for Site Improvement						$3,865,419
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	2.5	186	458	m^2	$4,056	$1,857,703
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
Bioreactor (HRAS)	4.3	375	1,620	m^2	$2,109	$3,416,744
Secondary Clarifiers	2.1	908	1,933	m^2	$2,644	$5,112,572
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.0	113	226	m^2	$20,905	$4,734,024
Digesters	1.0	3110	3,097	m^3	$1,030	$3,188,671
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR				m^3		$0
UV	-			ML/d PWWF		$0
Odour Control	Allowance					$130,000
Site Works:						
Pumping to Bioreactor			19	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1050mm dia.)		1050	258.5	m	$1,050	$271,425
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			25,230,834	%	4.00%	$1,009,233
Electrical substation		677	1	l.s.	$55,000	$55,000
Existing Facility Upgrades						$0
Sub-Total						**$26,295,067**
Division 1 Cost		2.5%				$560,741
Engineering		16%				$4,207,211
Project Management/ Quality Control		4%				$1,051,803
Contingency		30%				$7,888,520
Sub-total						**$40,003,342**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$40,003,342**

APPENDIX C: RESULTS OF SECOND LEVEL OF SCREENING

IIWWTP (Cost & Technical @ 50%)

Category	Evaluation Factor	Weighing	Iona Island Interim						Iona Island Build-out		
			Option 1A	Option 1B	Option 2	Option 3	Option 4	Option 5	Option 1	Option 2	Option 3
			50%CAS	100% CAS	50% RTF	50% HRAS	CEP+50% RTF	CEP	TF/SC	CAS	CEP+60% CAS
Cost	Capital	10%	7.4	3.3	7.9	7.4	7.4	10.0	9.3	8.2	10.0
	O/M	10%	9.4	7.9	10.0	9.4	6.1	6.7	10.0	9.4	7.3
	Lifecycle	10%	9.4	6.3	10.0	9.4	7.8	9.5	10.0	9.1	8.8
	Sub-total Cost	**30%**	**26.2**	**17.5**	**27.9**	**26.2**	**21.3**	**26.2**	**29.3**	**26.8**	**26.2**
Technical	Footprint	4%	2.4	0.7	3.2	3.0	3.0	4.0	4.0	2.7	3.7
	Ability to expand on site	0%									
	Ability to handle load variation	4%	2.4	2.8	2.8	2.4	4.0	3.2	3.2	3.2	4.0
	Ease of phasing	3%	2.5	3.0	2.5	2.0	1.5	1.0	2.4	2.4	3.0
	Ability to upgrade for N removal	3%	2.5	3.0	1.5	2.0	1.5	1.0	2.4	3.0	1.8
	Resiliency of process	3%	2.1	2.1	2.4	2.1	2.4	3.0	2.4	2.1	3.0
	Compatible with GVRD plants	3%	2.5	2.5	3.0	2.5	2.0	1.5	3.0	2.4	2.4
	Sub-Total Technical	**20%**	**14.4**	**14.1**	**15.4**	**14.0**	**14.4**	**13.7**	**17.4**	**15.8**	**17.9**
Environmental	Energy use	5%	**3.5**	**1.8**	**5.0**	**3.8**	**4.9**	**5.0**	**5.0**	**2.5**	**3.1**
	Greenhouse gases	4%	4.0	3.0	4.0	3.5	2.5	3.0	4.0	4.0	3.5
	Sludge production	4%	3.7	2.8	4.0	3.5	3.3	3.7	4.0	4.0	3.1
	Effluent quality	4%	2.5	4.0	3.0	2.0	2.5	2.0	4.0	4.0	3.0
	Wildlife habitat	4%	2.5	2.0	3.0	2.5	3.5	4.0	3.0	2.0	4.0
	Aerosols	4%	2.0	1.0	3.0	2.5	3.5	4.0	4.0	2.0	3.0
	Sub-total Sust/Env.	**25%**	**18.2**	**14.6**	**22.0**	**17.9**	**20.2**	**21.7**	**24.0**	**18.5**	**19.7**
Social	Visual impact	8%	6.9	6.9	4.6	6.9	4.6	8.0	4.6	6.9	8.0
	Risk of odours	9%	7.0	9.0	4.0	6.0	4.0	4.5	6.4	9.0	9.0
	Traffic generation	8%	7.0	6.0	8.0	6.5	5.0	4.5	8.0	8.0	4.0
	First nation concerns/issues	0%									
	Sub-total Social	**25%**	**20.9**	**21.9**	**16.6**	**19.4**	**13.6**	**17.0**	**19.0**	**23.9**	**21.0**
	Total		**79.7**	**68.2**	**81.9**	**77.5**	**69.5**	**78.7**	**89.6**	**85.0**	**84.7**
	Ranking		**2**	**6**	**1**	**4**	**5**	**3**	**1**	**3**	**2**

IIWWTP (Environment @ 50%)

Category	Evaluation Factor	Weighing	Iona Island Interim						Iona Island Build-out		
			Option 1A	Option 1B	Option 2	Option 3	Option 4	Option 5	Option 1	Option 2	Option 3
			50%CAS	100% CAS	50%RTF	50% HRAS	CEP+50% RTF	CEP	TF/SC	CAS	CEP+60% CAS
Cost	Capital	8%	5.9	2.7	6.3	5.9	5.9	8.0	7.4	6.6	8.0
	O/M	8%	7.5	6.3	8.0	7.5	4.9	5.4	8.0	7.6	5.9
	Lifecycle	8%	7.5	5.0	8.0	7.5	6.2	7.6	8.0	7.3	7.1
	Sub-total Cost	**24%**	**21.0**	**14.0**	**22.3**	**21.0**	**17.0**	**21.0**	**23.4**	**21.5**	**20.9**
Technical	Footprint	1%	0.6	0.2	0.8	0.8	0.7	1.0	1.0	0.7	0.9
	Ability to expand on site	0%									
	Ability to handle load variation	1%	0.6	0.7	0.7	0.6	1.0	0.8	0.8	0.8	1.0
	Ease of phasing	1%	0.8	1.0	0.8	0.7	0.5	0.3	0.8	0.8	1.0
	Ability to upgrade for N removal	1%	0.8	1.0	0.5	0.7	0.5	0.3	0.8	1.0	0.6
	Resiliency of process	1%	0.7	0.7	0.8	0.7	0.8	1.0	0.8	0.7	1.0
	Compatible with GVRD plants	1%	0.8	0.8	1.0	0.8	0.7	0.5	1.0	0.8	0.8
	Sub-Total Technical	**6%**	**4.4**	**4.4**	**4.6**	**4.2**	**4.2**	**4.0**	**5.2**	**4.8**	**5.3**
Environmental	Energy use	9%	**6.3**	**3.3**	**9.0**	**6.9**	**8.8**	**9.0**	**9.0**	**4.5**	**5.5**
	Greenhouse gases	9%	9.0	6.8	9.0	7.9	5.6	6.8	9.0	9.0	7.9
	Sludge production	8%	7.5	5.6	8.0	7.1	6.6	7.5	8.0	8.0	6.2
	Effluent quality	8%	5.0	8.0	6.0	4.0	5.0	4.0	8.0	8.0	6.0
	Wildlife habitat	8%	5.0	4.0	6.0	5.0	7.0	8.0	6.0	4.0	8.0
	Aerosols	8%	4.0	2.0	6.0	5.0	7.0	8.0	8.0	4.0	6.0
	Sub-total Sust/Env.	**50%**	**36.8**	**29.7**	**44.0**	**35.8**	**40.0**	**43.2**	**48.0**	**37.5**	**39.6**
Social	Visual impact	7%	6.0	6.0	4.0	6.0	4.0	7.0	4.0	6.0	7.0
	Risk of odours	7%	5.4	7.0	3.1	4.7	3.1	3.5	5.0	7.0	7.0
	Traffic generation	6%	5.3	4.5	6.0	4.9	3.8	3.4	6.0	6.0	3.0
	First nation concerns/issues	0%									
	Sub-total Social	**20%**	**16.7**	**17.5**	**13.1**	**15.6**	**10.9**	**13.9**	**15.0**	**19.0**	**17.0**
	TOTALS		**78.9**	**65.6**	**84.1**	**76.6**	**72.1**	**82.0**	**91.6**	**82.8**	**82.9**
	RANKING		**3**	**6**	**1**	**5**	**4**	**2**	**1**	**3**	**2**

IIWWTP (Social @ 50%)

Category	Evaluation Factor	Weighing	Iona Island Interim						Iona Island Build-out		
			Option 1A	Option 1B	Option 2	Option 3	Option 4	Option 5	Option 1	Option 2	Option 3
			50%CAS	100% CAS	50% RTF	50% HRAS	CEP+50% RTF	CEP	TF/SC	CAS	CEP+60% CAS
Cost	Capital	8%	5.9	2.7	6.3	5.9	5.9	8.0	7.4	6.6	8.0
	O/M	8%	7.5	6.3	8.0	7.5	4.9	5.4	8.0	7.6	5.9
	Lifecycle	8%	7.5	5.0	8.0	7.5	6.2	7.6	8.0	7.3	7.1
	Sub-total Cost	**24%**	**21.0**	**14.0**	**22.3**	**21.0**	**17.0**	**21.0**	**23.4**	**21.5**	**20.9**
Technical	Footprint	1%	0.6	0.2	0.8	0.8	0.7	1.0	1.0	0.7	0.9
	Ability to expand on site	0%	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Ability to handle load variation	1%	0.6	0.7	0.7	0.6	1.0	0.8	0.8	0.8	1.0
	Ease of phasing	1%	0.8	1.0	0.8	0.7	0.5	0.3	0.8	0.8	1.0
	Ability to upgrade for N removal	1%	0.8	1.0	0.5	0.7	0.5	0.3	0.8	1.0	0.6
	Resiliency of process	1%	0.7	0.7	0.8	0.7	0.8	1.0	0.8	0.7	1.0
	Compatible with GVRD plants	1%	0.8	0.8	1.0	0.8	0.7	0.5	1.0	0.8	0.8
	Sub-Total Technical	**6%**	**4.4**	**4.4**	**4.6**	**4.2**	**4.2**	**4.0**	**5.2**	**4.8**	**5.3**
Environmental	Energy use	4%	**2.8**	**1.5**	**4.0**	**3.1**	**3.9**	**4.0**	**4.0**	**2.0**	**2.5**
	Greenhouse gases	3%	3.0	2.3	3.0	2.6	1.9	2.3	3.0	3.0	2.6
	Sludge production	3%	2.8	2.1	3.0	2.7	2.5	2.8	3.0	3.0	2.3
	Effluent quality	3%	1.9	3.0	2.3	1.5	1.9	1.5	3.0	3.0	2.3
	Wildlife habitat	4%	2.5	2.0	3.0	2.5	3.5	4.0	3.0	2.0	4.0
	Aerosols	3%	1.5	0.8	2.3	1.9	2.6	3.0	3.0	1.5	2.3
	Sub-total Sust/Env.	**20%**	**14.5**	**11.6**	**17.5**	**14.2**	**16.2**	**17.6**	**19.0**	**14.5**	**15.9**
Social	Visual impact	17%	14.7	14.7	9.8	14.7	9.8	17.0	9.8	14.7	17.0
	Risk of odours	17%	13.2	17.0	7.6	11.3	7.6	8.5	12.1	17.0	17.0
	Traffic generation	16%	14.0	12.0	16.0	13.0	10.0	9.0	16.0	16.0	8.0
	First nation concerns/issues	0%									
	Sub-total Social	**50%**	**41.9**	**43.7**	**33.3**	**39.0**	**27.3**	**34.5**	**37.9**	**47.7**	**42.0**
	TOTALS		**81.7**	**73.6**	**77.8**	**78.4**	**64.8**	**77.0**	**85.5**	**88.4**	**84.2**
	RANKING		**1**	**5**	**2**	**4**	**6**	**3**	**2**	**1**	**3**

LGWWTP (Cost & Technical @ 50%)

Category	Evaluation Factor	Weighing	Lions Gate Interim					Lions Gate - Build-out				
			Option 1 50%BAF	Option 2A 50%RFT	Option 2B 100% RFT	Option 3 CEP+50% RTF	Option 4 HRAS	Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+60%TF/SC	Option 5 Pr + TF/SC
Cost	Capital	10%	6.4	10.0	8.2	8.9	N/A	9.4	9.8	N/A	10.0	8.4
	O/M	10%	8.7	10.0	9.5	7.6	N/A	10.0	9.7	N/A	7.7	10.0
	Lifecycle	10%	8.0	10.0	9.0	8.1	N/A	10.0	10.0	N/A	8.9	9.6
	Sub-total Cost	**30%**	**23.1**	**30.0**	**26.7**	**24.7**	**0.0**	**29.3**	**29.4**	**0.0**	**26.6**	**28.0**
Technical	Footprint	0%					N/A			N/A		
	Ability to expand on site	4%	4.0	0.0	0.0	1.6	N/A	0.0	4.0	N/A	1.6	0.8
	Ability to handle load variation	4%	3.2	4.0	4.0	2.8	N/A	4.0	3.2	N/A	2.8	4.0
	Ease of phasing	3%	2.4	1.8	1.8	3.0	N/A	1.8	2.4	N/A	3.0	1.8
	Ability to upgrade for N removal	3%	3.0	1.8	1.8	1.2	N/A	1.8	3.0	N/A	1.2	1.8
	Resiliency of process	3%	1.8	2.4	2.4	3.0	N/A	2.4	1.8	N/A	3.0	2.4
	Compatible with GVRD plants	3%	1.8	3.0	3.0	2.4	N/A	3.0	1.8	N/A	2.4	3.0
	Sub-Total Technical	**20%**	**16.2**	**13.0**	**13.0**	**14.0**	**0.0**	**13.0**	**16.2**	**0.0**	**14.0**	**13.8**
Environmental	Energy use	5%	3.9	5.0	4.4	4.4	N/A	5.0	3.9	N/A	4.1	5.0
	Greenhouse gases	5%	5.0	5.0	5.0	4.0	N/A	5.0	5.0	N/A	4.1	5.0
	Sludge production	5%	4.5	5.0	4.3	4.0	N/A	4.8	4.4	N/A	4.0	5.0
	Effluent quality	5%	2.5	2.5	5.0	3.9	N/A	5.0	5.0	N/A	4.0	5.0
	Wildlife habitat	0%					N/A			N/A		
	Aerosols	5%	3.0	4.0	4.0	5.0	N/A	4.0	3.0	N/A	5.0	4.0
	Sub-total Sust/Env.	**25%**	**18.8**	**21.5**	**22.7**	**21.3**	**0.0**	**23.8**	**21.3**	**0.0**	**21.2**	**24.0**
Social	Visual impact	6%	6.0	3.6	3.6	4.8	N/A	3.6	6.0	N/A	4.8	3.6
	Risk of odours	6%	6.0	3.6	3.6	4.8	N/A	3.6	6.0	N/A	4.8	3.6
	Traffic generation	6%	6.0	6.0	6.0	2.4	N/A	6.0	6.0	N/A	2.4	6.0
	First nation concerns/issues	7%	4.2	4.2	4.2	4.2	N/A	4.2	4.2	N/A	4.2	4.2
	Sub-total Social	**25%**	**22.2**	**17.4**	**17.4**	**16.2**	**0.0**	**17.4**	**22.2**	**0.0**	**16.2**	**17.4**
	TOTALS		**80.3**	**81.9**	**79.8**	**76.2**	**0.0**	**83.5**	**89.1**	**0.0**	**78.0**	**83.2**
	RANKING		**2**	**1**	**3**	**4**		**2**	**1**			**3**

LGWWTP (Environment @ 50%)

Category	Evaluation Factor	Weighing	Lions Gate Interim					Lions Gate - Build-out				
			Option 1 50%BAF	Option 2A 50%RFT	Option 2B 100%RFT	Option 3 CEP+50% RTF	Option 4 HRAS	Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+60%TF/SC	Option 5 Pr + TF/SC
Cost	Capital	8%	5.1	8.0	6.5	7.2	N/A	7.5	7.8	N/A	8.0	6.7
	O/M	8%	7.0	8.0	7.6	6.1	N/A	8.0	7.7	N/A	6.2	8.0
	Lifecycle	8%	6.4	8.0	7.2	6.5	N/A	8.0	8.0	N/A	7.1	7.7
	Sub-total Cost	**24%**	**18.5**	**24.0**	**21.3**	**19.7**	**0.0**	**23.5**	**23.5**	**0.0**	**21.3**	**22.4**
Technical	Footprint	0%					N/A			N/A		
	Ability to expand on site	1%	1.0	0.0	0.0	0.4	N/A	0.0	1.0	N/A	0.4	0.2
	Ability to handle load variation	1%	0.8	1.0	1.0	0.7	N/A	1.0	0.8	N/A	0.7	1.0
	Ease of phasing	1%	0.8	0.6	0.6	1.0	N/A	0.6	0.8	N/A	1.0	0.6
	Ability to upgrade for N removal	1%	1.0	0.6	0.6	0.4	N/A	0.6	1.0	N/A	0.4	0.6
	Resiliency of process	1%	0.6	0.8	0.8	1.0	N/A	0.8	0.6	N/A	1.0	0.8
	Compatible with GVRD plants	1%	0.6	1.0	1.0	0.8	N/A	1.0	0.6	N/A	0.8	1.0
	Sub-Total Technical	**6%**	**4.8**	**4.0**	**4.0**	**4.3**	**0.0**	**4.0**	**4.8**	**0.0**	**4.3**	**4.2**
Environmental	Energy use	10%	7.7	10.0	8.8	8.9	N/A	10.0	7.8	N/A	8.0	10.0
	Greenhouse gases	10%	10.0	10.0	10.0	8.0	N/A	10.0	10.0	N/A	8.0	10.0
	Sludge production	10%	8.9	10.0	8.7	8.1	N/A	9.5	8.8	N/A	8.0	10.0
	Effluent quality	10%	5.0	5.0	10.0	7.7	N/A	10.0	10.0	N/A	8.0	10.0
	Wildlife habitat	0%	0.0	0.0	0.0	0.0	N/A	0.0	0.0	N/A	0.0	0.0
	Aerosols	10%	6.0	8.0	8.0	10.0	N/A	8.0	6.0	N/A	10.0	8.0
	Sub-total Sust/Env.	**50%**	**37.6**	**43.0**	**45.5**	**42.7**	**0.0**	**47.5**	**42.6**	**0.0**	**42.0**	**48.0**
Social	Visual impact	5%	5.0	3.0	3.0	4.0	N/A	3.0	5.0	N/A	4.0	3.0
	Risk of odours	5%	5.0	3.0	3.0	4.0	N/A	3.0	5.0	N/A	4.0	3.0
	Traffic generation	5%	5.0	5.0	5.0	2.0	N/A	5.0	5.0	N/A	2.0	5.0
	First nation concerns/issues	5%	3.0	3.0	3.0	3.0	N/A	3.0	3.0	N/A	3.0	3.0
	Sub-total Social	**20%**	**18.0**	**14.0**	**14.0**	**13.0**	**0.0**	**14.0**	**18.0**	**na**	**13.0**	**14.0**
	TOTALS		**78.9**	**85.0**	**84.8**	**79.7**	**0.0**	**89.0**	**88.9**	**na**	**80.6**	**88.6**
	RANKING		**4**	**1**	**2**	**3**		**1**	**2**		**4**	**3**

LGWWTP (Social @ 50%)

Category	Evaluation Factor	Weighing	Lions Gate Interim					Lions Gate - Build-out				
			Option 1 50%BAF	Option 2A 50%RFT	Option 2B 100% RFT	Option 3 CEP+50% RTF	Option 4 HRAS	Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+60%TF/SC	Option 5 Pr + TF/SC
Cost	Capital	8%	5.1	8.0	6.5	7.2	N/A	7.5	7.8	N/A	8.0	6.7
	O/M	8%	7.0	8.0	7.6	6.1	N/A	8.0	7.7	N/A	6.2	8.0
	Lifecycle	8%	6.4	8.0	7.2	6.5	N/A	8.0	8.0	N/A	7.1	7.7
	Sub-total Cost	**24%**	**18.5**	**24.0**	**21.3**	**19.7**	**0.0**	**23.5**	**23.5**	**0.0**	**21.3**	**22.4**
Technical	Footprint	0%	0.0				N/A			N/A		
	Ability to expand on site	1%	1.0	0.0	0.0	0.4	N/A	0.0	1.0	N/A	0.4	0.2
	Ability to handle load variation	1%	0.8	1.0	1.0	0.7	N/A	1.0	0.8	N/A	0.7	1.0
	Ease of phasing	1%	0.8	0.6	0.6	1.0	N/A	0.6	0.8	N/A	1.0	0.6
	Ability to upgrade for N removal	1%	1.0	0.6	0.6	0.4	N/A	0.6	1.0	N/A	0.4	0.6
	Resiliency of process	1%	0.6	0.8	0.8	1.0	N/A	0.8	0.6	N/A	1.0	0.8
	Compatible with GVRD plants	1%	0.6	1.0	1.0	0.8	N/A	1.0	0.6	N/A	0.8	1.0
	Sub-Total Technical	**6%**	**4.8**	**4.0**	**4.0**	**4.3**	**0.0**	**4.0**	**4.8**	**0.0**	**4.3**	**4.2**
Environmental	Energy use	4%	3.1	4.0	3.5	3.6	N/A	4.0	3.1	N/A	3.2	4.0
	Greenhouse gases	4%	4.0	4.0	4.0	3.2	N/A	4.0	4.0	N/A	3.2	4.0
	Sludge production	4%	3.6	4.0	3.5	3.2	N/A	3.8	3.5	N/A	3.2	4.0
	Effluent quality	4%	2.0	2.0	4.0	3.1	N/A	4.0	4.0	N/A	3.2	4.0
	Wildlife habitat	0%	0.0	0.0	0.0	0.0	N/A	0.0	0.0	N/A	0.0	0.0
	Aerosols	4%	2.4	3.2	3.2	4.0	N/A	3.2	2.4	N/A	4.0	3.2
	Sub-total Sust/Env.	**20%**	**15.0**	**17.2**	**18.2**	**17.1**	**0.0**	**19.0**	**17.0**	**0.0**	**16.8**	**19.2**
Social	Visual impact	12%	12.0	7.2	7.2	9.6	N/A	7.2	12.0	N/A	9.6	7.2
	Risk of odours	13%	13.0	7.8	7.8	10.4	N/A	7.8	13.0	N/A	10.4	7.8
	Traffic generation	12%	12.0	12.0	12.0	4.8	N/A	12.0	12.0	N/A	4.8	12.0
	First nation concerns/issues	13%	7.8	7.8	7.8	7.8	N/A	7.8	7.8	N/A	7.8	7.8
	Sub-total Social	**50%**	**44.8**	**34.8**	**34.8**	**32.6**	**0.0**	**34.8**	**44.8**	**0.0**	**32.6**	**34.8**
	TOTALS		**83.1**	**80.0**	**78.3**	**73.7**	**0.0**	**81.3**	**90.1**	**0.0**	**75.0**	**80.6**
	RANKING		**1**	**2**	**3**	**4**		**2**	**1**		**4**	**3**

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 4
Consideration of Build-Out to Secondary Treatment

FINAL REPORT

Prepared for

Greater Vancouver Regional District

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1 INTRODUCTION

This report (Appendix 4) describes the requirements for build-out to secondary at Iona Island and Lions Gate wastewater treatment plants. Under the approved Liquid Waste Management Plan (LWMP), upgrading to full secondary treatment is required by 2021 for Iona Island WWTP and by 2031 at Lions Gate WWTP.

This report should be read in conjunction with Appendix 3 which describes the requirements for interim upgrades.

Appendix 3 and 4 should also be read in conjunction with Appendix 10. Essentially Appendix 10 is the continuation of Appendix 3 and 4. Appendix 3 and 4 describe the analysis for the first and the second levels of screening where the number of options for interim upgrades was reduced from 27 to 4 or 5 and the number of options for upgrade to secondary was reduced from 14 to 1 or 2.

The short list of options identified in Appendix 3 and 4 are then examined in more detail in Appendix 10 including revised cost estimates. The cost estimates in Appendix 3 and 4 were developed as tools for the screening of options. For more accurate cost estimates, the reader should refer to Appendix 10.

Also while the work covered by Appendix 3 and 4 was under way, the reports for Appendix 7 (Interim Sludge Handling) and Appendix 8 (Condition of Existing Treatment Plant) were being developed. The recommendations of the reports for Appendix 7 and 8 were carried over in the report for Appendix 10.

Finally, the reports for Appendix 1 to 10 are brought together in the Summary Report

The proposed effluent criteria for a secondary treatment plant are described in Section 2. Detailed flow and load projections were carried out in order to generate a lower and upper envelope. Separate flow and load projections were prepared for the various contributors. The contributors are: (1) residential, (2) commercial and institutional, (3) industrial, (4) groundwater infiltration and (5) trucked liquid waste. For each contributor, lower and upper growth rates were established and the impact of various scenarios for source control were estimated. Lower and upper envelopes for flows and loads were prepared by adding the lower and upper envelopes for the five above components. The methodology for flow and load projections are described in detail in Appendix # 3 and are summarized in Section 3.

For both plants, a comprehensive number of options for build-out to secondary were identified. These include (1) fixed-film processes, (2) suspended growth processes, (3) anaerobic processes, (4) combination fixed film and suspended growth, (5) chemical oxidation, (6) primary treatment followed by partial biological treatment and (7) chemically enhanced primary (CEP) treatment followed by partial biological treatment. Each treatment process is described in Section 4.

All the options described in Section 4 were screened and ranked using a two-step approach. All options were initially screened using pass or fail criteria. Those options that passed were then ranked. This first level of screening and ranking is described in Section 7.

2 OBJECTIVES OF SECONDARY TREATMENT

2.1 MUNICIPAL SEWAGE REGULATION

The Liquid Waste Management Plan does not contain a definition of secondary treatment for the Iona Island WWTP and Lions Gate WWTP nor does it include effluent criteria. The ***BC Municipal Sewage Regulation*** includes the following definition of secondary treatment:

> ***Secondary treatment – any form of treatment, excluding dilution, that consistently produces an effluent quality with a BOD$_5$ not exceeding 45 mg/L and TSS not exceeding 45 mg/L for flows up to 2.0 x ADWF.***

The B.C. Municipal Sewage Regulation requires monitoring and sampling as follows:

Monitoring of effluent quality shall be undertaken 5 times per week using a 24 Hr flow proportional composite sample for plants with maximum flows in excess of 200,000 m3/day. Sampling frequency is reduced to 2 times per week for plants with maximum flows in the range of 50,000 to 200,000 m^3/day.

2.2 US EPA

The US EPA has the following effluent limits for secondary treatment:

- 30-day average
  a) BOD$_5$: < 30 mg/L
  b) TSS: < 30 mg/L
  c) pH: 6.0-9.0
  d) 85% removal efficiency for BOD$_5$ and TSS

- 7-day average
  a) BOD$_5$: < 45 mg/L
  b) TSS: < 45 mg/L

- Equivalent to secondary permit limits

  Permit for equivalent-to-secondary facility selected by the State from 30 to 45 mg/L of BOD$_5$ and TSS 30-day average and from 45 to 65 of BOD$_5$ and TSS for 7-day average.

- Alternative State Requirement

  States have the flexibility to set permit limits above the maximum level of 45 mg/L 30-day average and 65 mg/L 7-day average BOD$_5$ and TSS.

2.3 DRAFT FEDERAL POLICY ON AMMONIA

The draft Federal policy on ammonia which was published in June 2003 applies to wastewater treatment systems where the annual average effluent release during 2004 from that system to surface water is greater than or equal to 5,000 m^3 per day and where any of the following three conditions are met:

(1) Residual Chlorine

The concentration of total residual chlorine (TRC) in the release exceeded 0.02 mg/L at any time during 2004.

(2) Ammonia and Depth of Outfall

(a) The concentration of ammonia nitrogen (NH_3-N) in the effluent exceeded 16 mg/L at any time during the period of June 1, 2004, to September 30, 2004; and

(b) The depth of water over the effluent release point, at any time during the period of June 1, 2004, to September 30, 2004, is less than 15 times the diameter of the discharge pipe or the diameter of a diffuser port in the discharge pipe.

(3) Ammonia, pH and Fresh Water

(a) The effluent release is to fresh water; and

(b) The concentration of ammonia nitrogen (NH_3-N) in the effluent exceeded 16 mg/L at any time during the period of June 1, 2004, to September 30, 2004; and

(c) The pH of the surface water upstream of the effluent release point exceeded 7.5 at any time during the period of June 1, 2004, to September 30, 2004.

Since the effluent from Iona Island and Lions Gate WWTP does not contain residual chlorine, is discharged at depths greater than 15 times the diameter of the discharge pipe and is not released to fresh water, the proposed Federal policy is not applicable.

3 SUMMARY OF DESIGN FLOWS AND LOADS

3.1 IONA ISLAND

Under the Liquid Waste management Plan (LWMP), full secondary treatment upgrade at Iona Island WWTP is required by 2021. It is proposed that secondary treatment will be provided for flows up to two times average dry weather flow (ADWF). For the purpose of this study, ADWF and maximum monthly (MM) loads projected to the year 2036 will be used to design the secondary treatment facility at IIWWTP. Flow in excess of two times ADWF will receive primary treatment only.

Figures 3.1 to 3.3 present graphical representations of the lower and upper envelopes and the design case scenario for Average Dry Weather Flow (ADWF), BOD (Max. Month) and TSS (Max. Month). The lower and upper envelopes take into consideration of the impacts of future conditions, population growth ranges in the Vancouver Sewerage Area (VSA), and various Demand Side management (DSM) scenarios on flow and loads. The methodology to arrive at the upper and lower envelopes and design case scenario is discussed in detail in Appendix 3. Figure 3.4 presents the base case projection of ADWF for IIWWTP and the combined flow of both IIWWTP and LGWWTP following secondary treatment upgrades at IIWWTP.

The GVRD 2001 report provided the maximum value of the MM/AA ratios measured over the period of data analyzed. These were used in making the projections which are represented by the lines. One would therefore expect that the start of the projected lines would be above all of the annual maximum month values (spot values) and would start at near the maximum spot value.

Table 3.1 summarizes the design flows and loads used in various options of the secondary build-out treatment process design for the IIWWTP. The design case scenario of flows and loads for the final option is used. The combined flows and loads consist of the base case projections at IIWWTP and the upper projection envelopes at LGWWTP.

TABLE 3.1
YEAR 2036 SECONDARY BUILD-OUT DESIGN FLOWS AND LOADS

Design Parameter	**IIWWTP Upper Envelope**	**IIWWTP and LGWWTP Combined**
Average Dry Weather Flow (ML/d), ADWF	500	616
Average Annual Flow (ML/d), AAF	650	789
Peak Wet Weather Flow (ML/d), PWWF	1,530	1,908
Maximum Month BOD Loading (t/d), MM BOD	124	150
Maximum Month TSS Loading (t/d), MM TSS	113	144

FIGURE 3.1
GRAPHICAL REPRESENTATIONS OF THE LOWER, UPPER ENVELOPES AND THE DESIGN CASE SCENARIOS FOR ADWF

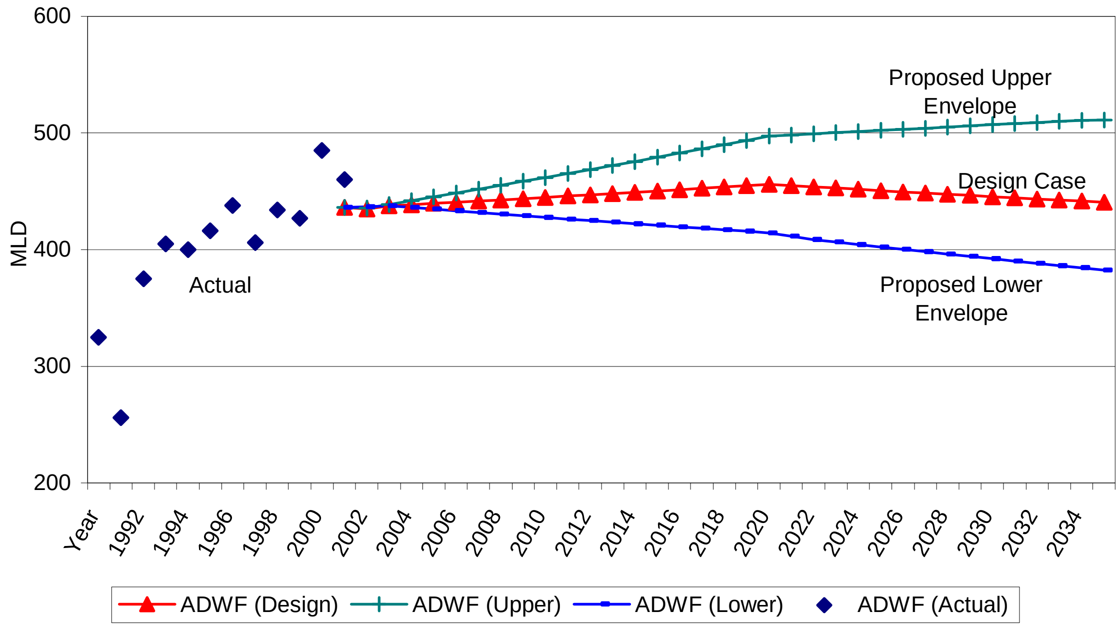


FIGURE 3.2
GRAPHICAL REPRESENTATIONS OF THE LOWER, UPPER ENVELOPES AND THE DESIGN CASE SCENARIOS FOR BOD

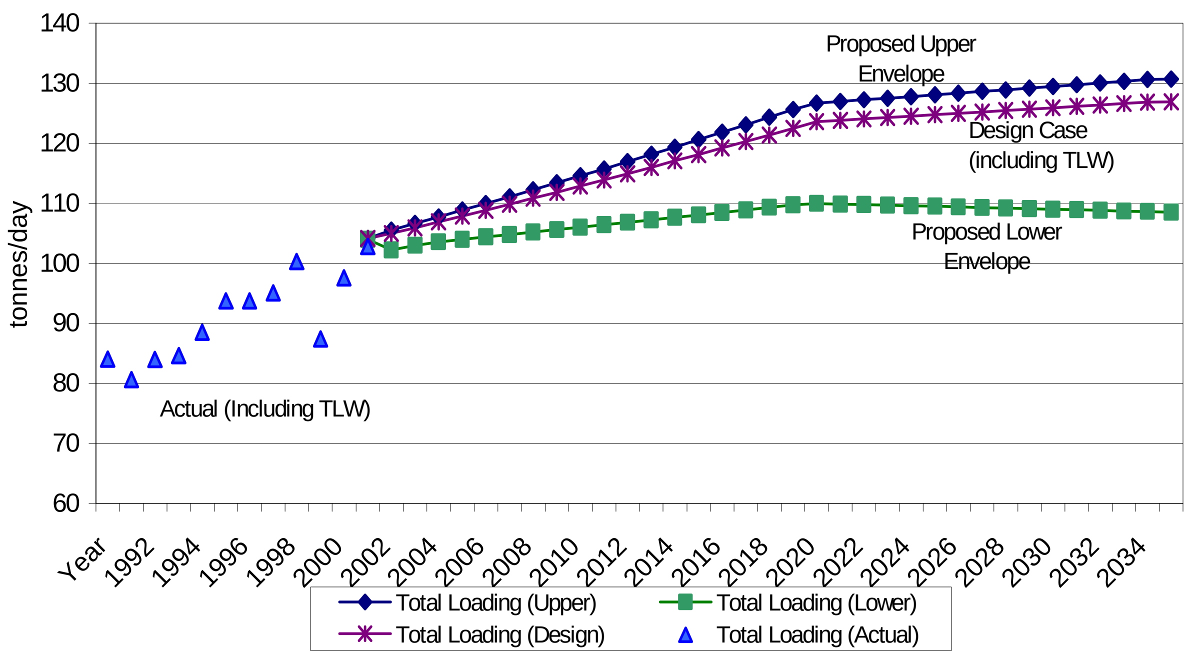


FIGURE 3.3
GRAPHICAL REPRESENTATIONS OF THE LOWER, UPPER ENVELOPES AND THE DESIGN CASE SCENARIOS FOR TSS

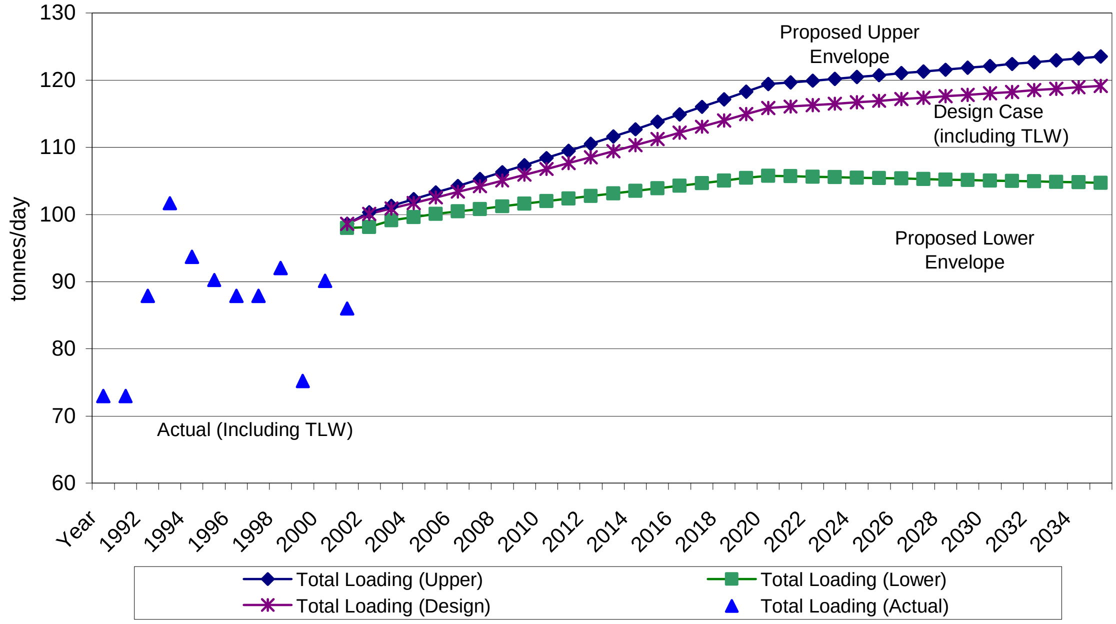
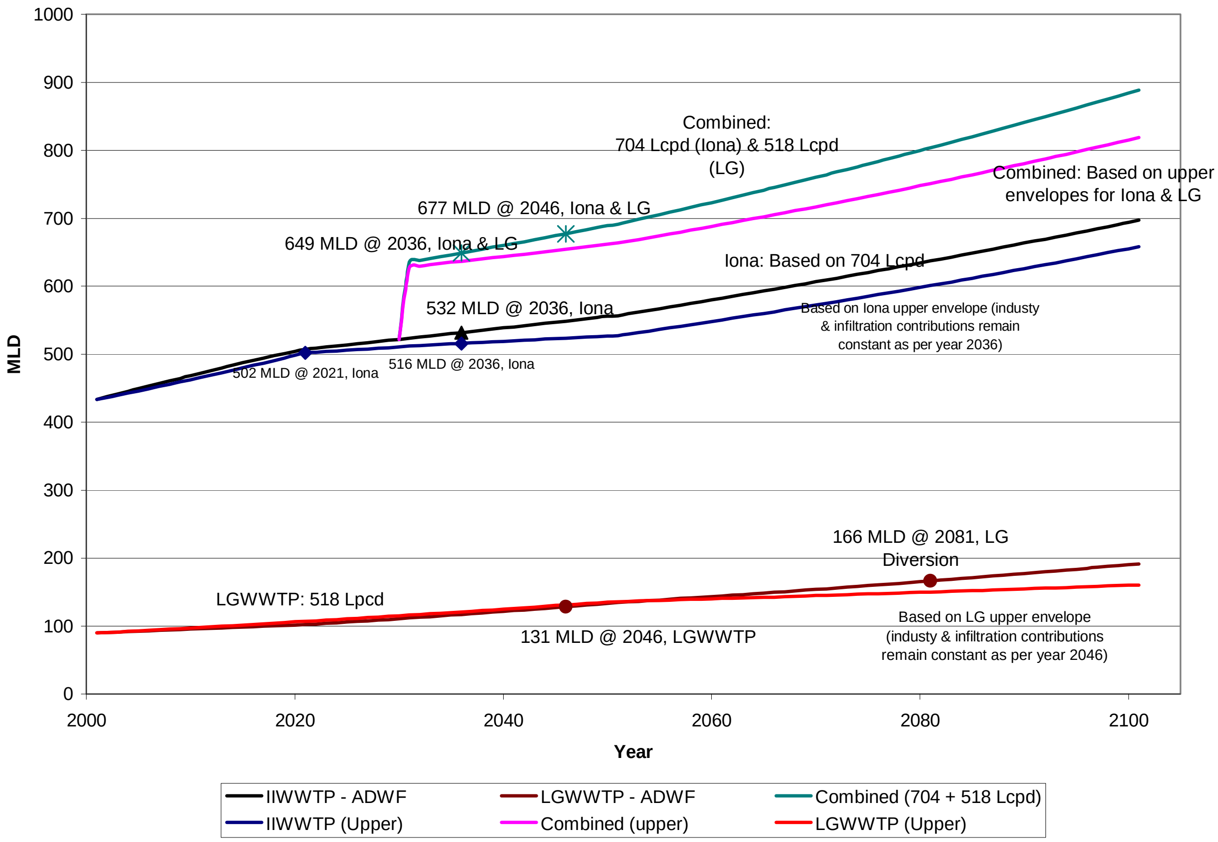


FIGURE 3.4
THE BASECASE PROJECTION OF ADWF FOR IIWWTP



3.2 LIONS GATE

Similarly, under the Liquid Waste Management Plan (LWMP), full secondary treatment upgrade at Lions Gate WWTP is required by 2031. It is proposed to provide secondary treatment for flows up to two times average dry weather flow (ADWF) for build-out. For the purpose of this study, ADWF and maximum monthly (MM) loads projected to the year 2046 will be used to design the secondary treatment facility at LGWWTP.

Figures 3.5 to 3.8 present graphs of historical data (1991-2003) and the projections of the lower envelope, upper envelope, and the design case scenario for Average Dry Weather Flow (ADWF), BOD (Max. Month) and TSS (Max. Month) at LGWWTP. The projections take into consideration the impacts of future conditions, population growth in the North Shore Sewerage Area (NSSA), and the impact of various Demand Side management (DSM) scenarios on flow and loads. The methodology to arrive at the upper, lower envelopes and design case scenario is discussed in detail in Appendix # 3.

The GVRD 2001 report assumed the maximum value of the MM/AA ratios measured over the period of data analyzed. These were used in making the projections which are represented by the lines. One would therefore expect that the start of the projected lines would be above all of the annual maximum month values (spot values) and would start at near the maximum spot value. However it appears that the maximum spot value was associated with a larger than normal annual average load which results in a MM/AA ratio different from the average. The data variations illustrate the difficulty of predicting maximum values.

Table 3.2 summarizes the design flows and loads used in the various initial options of the secondary build-out treatment process design for the LGWWTP. The design case scenario of flows and loads for the final option is used.

TABLE 3.2
YEAR 2046 SECONDARY BUILD-OUT DESIGN FLOWS AND LOADS LGWWTP

Design Parameter	Upper Envelope	Design Case
Average Dry Weather Flow (ML/d), ADWF	131	111
Average Annual Flow (ML/d), AAF	157	133
Peak Wet Weather Flow (ML/d), PWWF	420	336
Maximum Month BOD Loading (t/d), MM BOD	30	28
Maximum Month TSS Loading (t/d), MM TSS	36	32

FIGURE 3.5
LGWWTP GRAPH OF THE LOWER, UPPER ENVELOPES AND THE DESIGN CASE SCENARIOS FOR ADWF

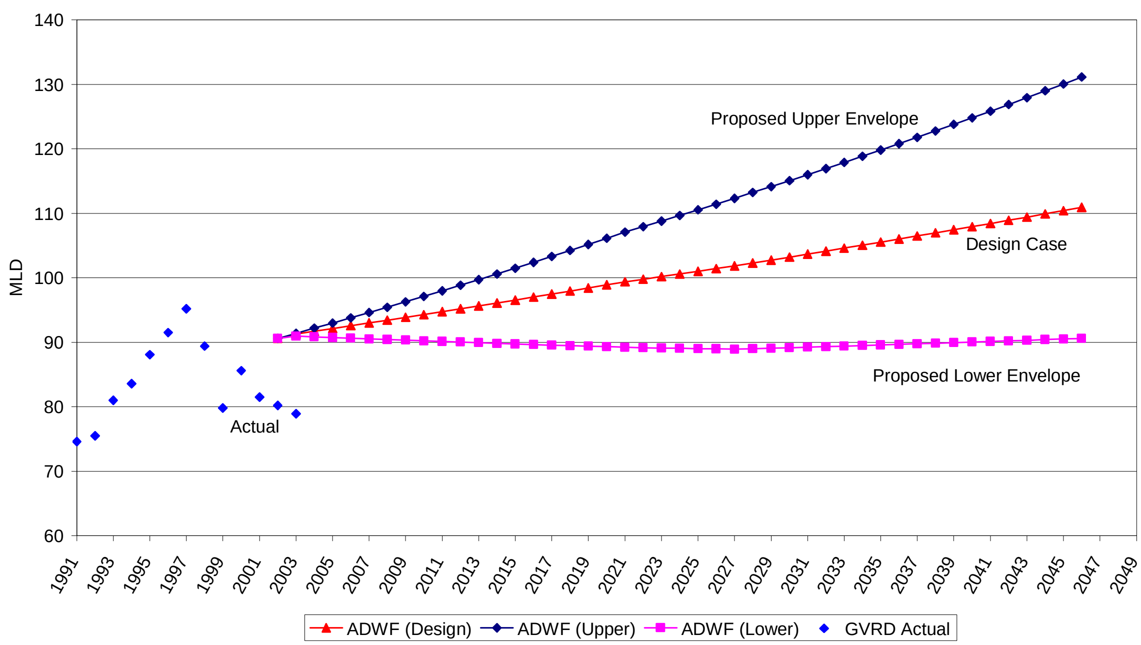


FIGURE 3.6
LIONS GATE WWTP GRAPH OF THE LOWER, UPPER ENVELOPE, AND THE DESIGN CASE SCENARIO FOR BOD (MAX. MONTH)

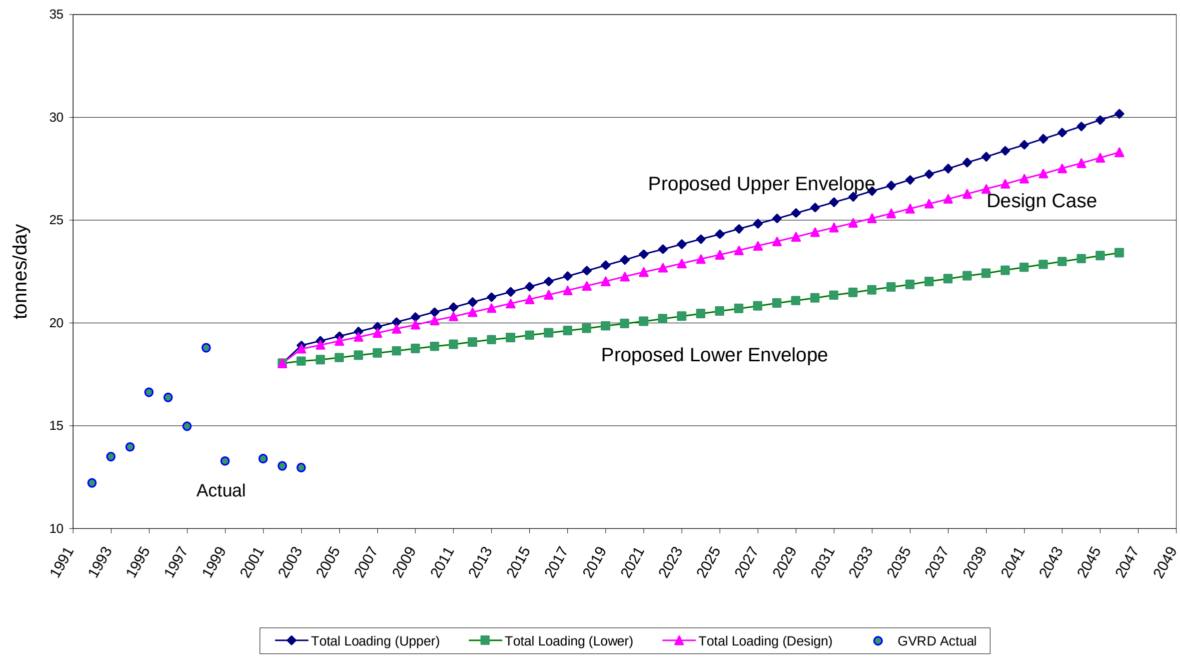
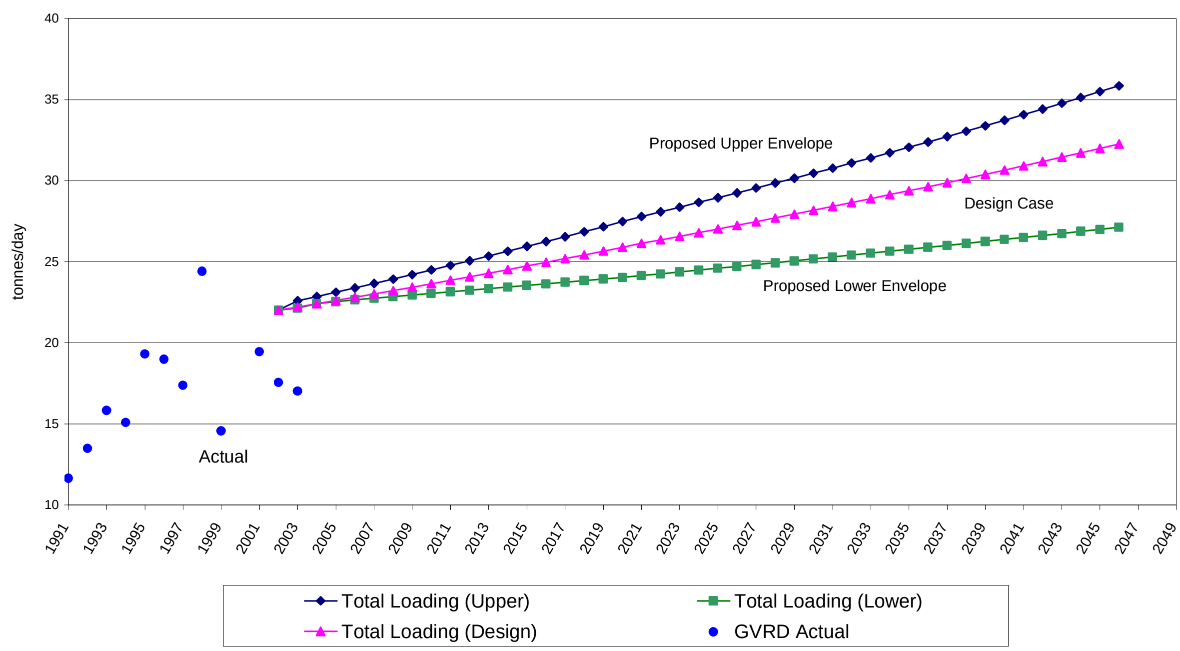


FIGURE 3.7
LIONS GATE WWTP GRAPH OF THE LOWER, UPPER ENVELOPE, AND THE DESIGN CASE SCENARIO FOR TSS (MAX. MONTH)



4 ALTERATIVES FOR SECONDARY TREATMENT

4.1 FIXED FILM PROCESSES

4.1.1 Roughing Trickling Filter (RTF)

4.1.1.1 Process Description
(WEF MOP 8, 4th Edition)

A summary description and process diagram of a roughing or ultra high rate trickling filter is provided in Figure 4.1. RTFs support hydraulic loading rates of 11.7-70 m^3/m^2-d. Ultra high-rate can support hydraulic loading rates of 47 – 188 m^3/m^2-d. Organic loading of BOD_5 is also high at 0.5-1.6 kg/m^3-d and 1.6-8 kg/m^3-d, respectively.

A common method of upgrading existing activated-sludge plants is to install a roughing filter ahead of the activated-sludge process. As part of the roughing trickling filter activated sludge (RTF/AS) process, the roughing filter is typically 15 to 30% of the size required if treatment had been accomplished through the use of the trickling filter process alone. Hydraulic retention time in the aeration basin is typically 35 to 50% that required with the use of the activated-sludge process alone.

Some TF plants have been built to operate with two or more TF units in series. These plants are called two-stage or multistage TF plants if intervening clarification is included. Two filters directly coupled in series and operated at the same hydraulic rates typically perform as if they were one unit of the same diameter with the total depth of the two filters, especially if they have forced ventilation.

Under current practice, distinctions are made among TF applications based on the treatment provided rather than the hydraulic rate or organic loading of the application. This approach more accurately identifies the purpose of the TF operation. Hence, the general types of TFs are:

- Roughing filters that provide approximately 50 to 75% SBOD removal and 30 to 45% BOD5 oxidation, followed by a second stage of treatment;
- Complete treatment filters that provide the required settled effluent BOD5 and TSS;
- Combined BOD5 removal and NOD removal filters that provide the required settled effluent quality for BOD5, TSS, and ammonium-nitrogen; and
- Tertiary nitrifying filters that provide required effluent ammonium-nitrogen in a tertiary mode receiving a clarified secondary influent.

Process Description

(See also Trickling Filter - Standard Rate) Roughing filters are specially designed trickling filters, typically operated at high hydraulic loadings, necessitating the use of high recycle rates. They are used primarily to reduce organic loading on downstream processes and in seasonal nitrification applications. As with other biological processes, roughing-filter performance is temperature-sensitive. The higher hydraulic loadings of this kind of filter cause nearly continuous sloughing of the slime layer. If unsettled filter effluent is used for recycle, the sloughed biological solids in the recycle stream may contribute to organic removal within the filter as in a suspended-growth process.

Design Criteria

Filter medium: Plastic / redwood / cedar
Hydraulic loading: 11.7 - 70 $m^3/m^2.d$ (Ultra High-Rate)
47 - 188 $m^3/m^2.d$ (Roughing)
Organic loading: 0.5 - 1.6 $kg/m^3.d$ (Ultra High-Rate)
1.6 - 8 $kg/m^3.d$ (Roughing)
Depth: 0.9 - 6m
Recirculation ratio: 1 - 4
Sloughing: continuous
Power: 8 - 16 kW/ML

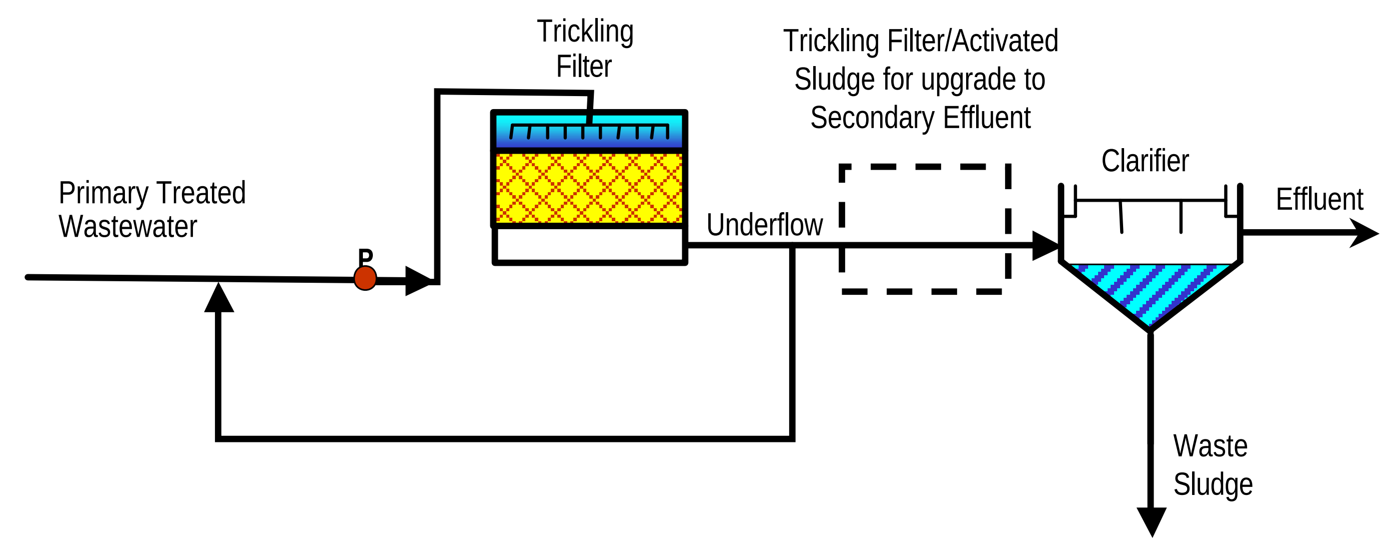
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	40% - 70%
TSS mg/L	70% - 80%

Plant Footprint

Can be accommodated on the Iona and Lions Gate sites

Schematics



Advantages

- Proven Technology
- Operating Costs Lower Than AS
- Simple operation
- Robust process resistant to toxic and hydraulic shocks
- Low energy requirement for aeration
- Biomass cannot be washed out by high peak flows
- Does not suffer from filamentous bacteria, sludge bulking, foaming
- Biomass has excellent settling qualities
- Smaller footprint than activated sludge
- Potential for partial treatment of full flow for MBAS removal

Disadvantages

- Possible psychoda (filterfly) nuisance
- Odour potential from trickling filter
- Can have High trickling filter profile (visual)
- Poorer quality effluent than TF/SC or activated sludge

Comment

- Interim a roughing filter could provide economy by acting as partial treatment of the full flow and later as the first biological treatment component at either Iona Island or Lions Gate. In the case of Lions Gate, the small footprint would be advantageous.

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Adequately sized final settling tanks are required to achieve proper effluent TSS and BOD_5 levels. Application of modern and deeper clarifier designs with energy-dissipating, center-feed wells, baffled launders, and moderate overflow rates are keys to good effluent quality.

4.1.1.2 Proven Technology
(WEF MOP 8, 4th Edition)

Technologies currently available can produce Advanced Wastewater Treatment (AWT) effluents of 10 mg/L BOD_5 and TSS or less and ammonium-nitrogen effluents of 1 mg/L or less. Trickling filters have historically been considered vulnerable to climatic changes because wastewater droplets must be exposed to large volumes of ambient-temperature air. However, proper engineering design can reduce temperature losses caused by wind and ventilation to less than 1.5°C. Improving dosing procedures and minimizing recirculation can also help control temperature loss.

Temperature effects on nitrifying trickling filters are now considered to be less significant than those on activated sludge. Earlier observations of poor effluent quality in winter were caused by a combination of shallow filters with high surface area, low freeboard, and high recirculation ratios that caused excessive heat losses. Other conditions contributing to poor performance included poor clarifier designs and filter dosing procedures that caused excess solids accumulations.

Trickling filters are no longer viewed only as a process to produce secondary treatment effluent. The TF process now used for AWT produces low residual BOD_5, TSS, and ammonium-nitrogen. Replacing existing TFs is often more expensive than updating and expanding existing units using known process technology such as the addition of short-term aeration or the solids-contact process.

In applications where more stringent effluent quality standards have exceeded the capability of existing TF designs, expanding TF capabilities often meets the requirements. Based on recent experiences, the full potential of the TF is only now being realized. The improved treatment capabilities of new and modified facilities, along with inherent ease of operation and low power use, have resulted in continued use of TFs.

4.1.1.3 Discharge Requirements/Effluent Quality

If properly designed, high rate TF process will reliably achieve the LWMP treatment requirements of 45 mg/L BOD_5 and 45 mg/L TSS with a conservative design and power ventilation and recirculation. In order to achieve better effluent quality, TF process must be combined with suspended growth process such as solids contact tanks.

4.1.1.4 Reliability

Successful conventional secondary and AWT applications are achievable with TFs but require a better understanding of TF operation and required appurtenances. If proper design procedures are used, TF performance equaling that of suspended-growth systems can be achieved:

- Trickling filters can produce effluent qualities of <10 mg/L BOD5 and TSS;
- The effluent can be comparable to activated-sludge effluent;
- Trickling filters rapidly reduce soluble BOD5 in applied wastewater;
- Temperature loss is less than 1.5°C in cold climates;
- Trickling filters are efficient nitrification units and effluents of <1.0 mg/L ammonium-nitrogen can be produced;
- Natural ventilation is inadequate for optimizing performance and power ventilation should be used;
- For rotating arm applicators, trickling filters should be dosed every 10 to 60 seconds, but routine flushing at 10 to 30 minutes/dose is also needed to enhance performance; alternatively, solid set and pumped application has wider flushing capability;
- Recirculation is typically beneficial for optimum performance, especially if the hydraulic loading rate is low;
- Power consumption is typically 25% less than activated-sludge treatment;
- Trickling filter sloughing cycles are harmful to filter performance and can be avoided by daily flushing; and
- Less land area is required for TFs than for activated-sludge treatment.

4.1.1.5 Site Suitability

The Ultra High rate TF facility is suited to either the Iona or the Lions Gate sites. For Iona, the site is of sufficient size to easily accommodate the TF. For either site the TF process could be added as a downstream process to the primary clarifiers. At Lions Gate, the TF addition may need to incorporate high towers.

4.1.2 Standard Rate Trickling Filter

4.1.2.1 Process Description
(WEF MOP 8, 4th edition)

A summary description and process diagram are provided in Figure 4.2. Before 1936, only low-rate (also called standard-rate) TFs had been constructed in the U.S. Design hydraulic loadings ranged from 0.018 to 0.036 L/s-m^2, or 0.08 to 0.16 m^3/m^2-h. Organic loading of BOD_5 is about 0.08 – 0.16 kg/m^3-d. These are typically constructed of rock and are low profile. The intermediate rate TF's are constructed for 2-3 times the standard rate loading.

Some TF plants have been built to operate with two or more TF units in series. These plants are called two-stage or multistage TF plants if intervening clarification is included. Two filters directly coupled in series and operated at the same hydraulic rates typically perform as if they were one unit of the same diameter with the total depth of the two filters, especially if they have forced ventilation.

Under current practice, distinctions are made among TF applications based on the treatment provided rather than the hydraulic rate or organic loading of the application. This approach more accurately identifies the purpose of the TF operation. Hence, the general types of TFs are:

- Roughing filters that provide approximately 50 to 75% SBOD removal and 30 to 45% BOD_5 oxidation, followed by a second stage of treatment;
- Complete treatment filters that provide the required settled effluent BOD_5 and TSS;
- Combined BOD_5 removal and nitrification filters that provide the required settled effluent quality for BOD_5, TSS, and ammonium-nitrogen; and
- Tertiary nitrifying filters that provide required effluent ammonium-nitrogen in a tertiary mode when receiving a clarified secondary influent.

Adequately sized final settling tanks are required to achieve proper effluent levels of TSS and BOD_5. Application of modern and deeper clarifier designs with energy-dissipating, center-feed wells, baffled launders, and moderate overflow rates are keys to good effluent quality.

TRICKLING FILTER - STANDARD RATE

Process Description

The trickling filter process depends on biological activity to oxidize the complex organic matter in wastewater. Wastewater is distributed over a bed of fixed media and bacteria develop in a slime layer attached to the media. Entrainment of atmospheric oxygen occurs as the wastewater “trickles” down over the surface of the media. An underdrain system is provided to collect the treated liquid exiting the media bed. As the slime layer grows in thickness, hydraulic shear forces cause clumps of bacteria to “slough” from the media. Sloughed solids are separated from the treated liquid by gravity settling in the final clarifiers.

Design Criteria

Hydraulic loading: 2 - 4 $m^3/m^2.d$
Organic loading: 0.08 - 0.16 kg BOD/m^3.d
Recirculation ratio: 0
Sloughing: Intermittent
Depth: 1.8 - 2.4m
Power: 2 - 4 kW/ML

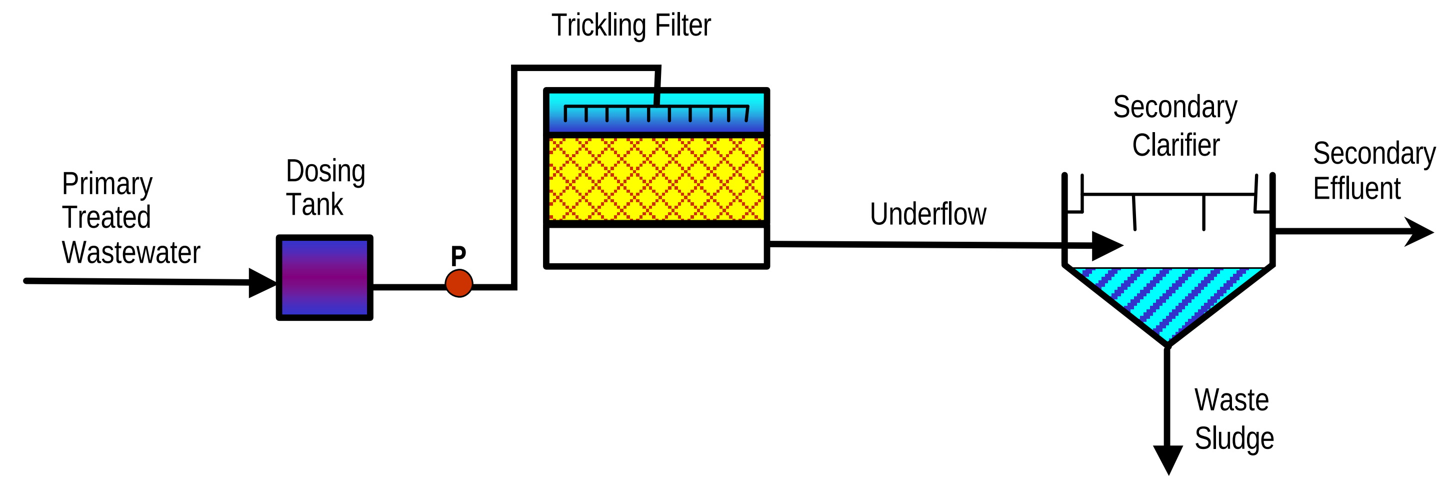
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	80% - 90%
TSS mg/L	80% - 90%

Plant Footprint

Can be accommodated on the Iona and Lions Gate sites

Schematics



Advantages

- Proven Technology
- Odour Control Limited
- Operating Costs Lower Than AS
- Simple operation
- Robust process resistant to toxic and hydraulic shocks
- Low energy requirement for aeration
- Biomass cannot be washed out by high peak flows
- Does not suffer from filamentous bacteria, sludge bulking, foaming
- Biomass has excellent settling qualities

Disadvantages

- Possible psychoda (filterfly) nuisance
- Odour potential from trickling filter
- Can have High trickling filter profile (visual)
- Poorer quality effluent than activated sludge

Comment

- very popular for use in small plants

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4.1.2.2 Proven Technology
(WEF MOP 8, 4th Edition)

Technologies currently available can produce Advanced Wastewater Treatment (AWT) effluents of 10 mg/L BOD5 and TSS or less and ammonium-nitrogen effluents of 1 mg/L or less. Trickling filters have historically been considered vulnerable to climatic changes because wastewater droplets must be exposed to large volumes of ambient-temperature air. However, proper engineering design can reduce temperature losses caused by wind and ventilation to less than 1.5°C (2.7°F). Improving dosing procedures and minimizing recirculation can also help control temperature loss.

Temperature effects on nitrifying trickling filters are now considered to be less significant than those on activated sludge. Earlier observations of poor effluent quality in winter were caused by a combination of shallow filters with high surface area, low freeboard, and high recirculation ratios that caused excessive heat losses. Other conditions contributing to poor performance included poor clarifier designs and filter dosing procedures that caused excess solids accumulations.

Trickling filters are no longer viewed only as a process to produce secondary treatment effluent. The TF process now used for AWT produces low residual BOD_5, TSS, and ammonium-nitrogen. Replacing existing TFs is often more expensive than updating and expanding existing units using known process technology such as the addition of short-term aeration or the solids-contact process.

In applications where more stringent effluent quality standards have exceeded the capability of existing TF designs, expanding TF capabilities often meets the requirements. Based on recent experiences, the full potential of the TF is only now being realized. The improved treatment capabilities of new and modified facilities, along with inherent ease of operation and low power use, have resulted in continued use of TFs.

4.1.2.3 Discharge Requirements/Effluent Quality

If properly designed, the TF process will reliably achieve the LWMP treatment requirements of 45 mg/L BOD_5 and 45 mg/L TSS with a conservative design and power ventilation and recirculation. In order to achieve better effluent quality, TF process must be combined with suspended growth process such as solids contact tanks. Should it be required, the TF reactors can be increased in height to provide ammonia removal through nitrification, but at increased cost.

4.1.2.4 Reliability

Successful conventional secondary and AWT applications are achievable with TFs but require a better understanding of TF operation and required appurtenances. If proper design procedures are used, TF performance equaling that of suspended-growth systems can be achieved:

- Trickling filters can produce effluent qualities of <10 mg/L BOD5 and TSS;
- The effluent can be comparable to activated-sludge effluent;
- Trickling filters rapidly reduce soluble BOD5 in applied wastewater;
- Temperature loss is less than 1.5°C in cold climates;
- Trickling filters are efficient nitrification units and effluents of <1.0 mg/L ammonium-nitrogen can be produced;
- Natural ventilation is inadequate for optimizing performance and power ventilation should be used;
- For rotating arm applicators, trickling filters should be dosed every 10 to 60 seconds, but routine flushing; at 10 to 30 minutes/dose is also needed to enhance performance; alternatively, solid set and pumped application has wider flushing capability;
- Recirculation is typically beneficial for optimum performance, especially if the hydraulic loading rate is low;
- Power consumption is typically 25% less than activated-sludge treatment;
- Trickling filter sloughing cycles are harmful to filter performance and can be avoided by daily flushing; and
- Less land area is required for TFs than for activated-sludge treatment.

4.1.2.5 Site Suitability

The standard rate TF facility is suited to either the Iona or the Lions Gate sites. For Iona, the site is of sufficient size to easily accommodate the TF. For the Lions Gate site the TF process could be added as a downstream process to the primary clarifiers. The use of a standard rate TF is not as appropriate as other TF designs and higher rate options would be used today over the standard or intermediate rate TF.

4.1.3 Trickling Filter-Solids Contact

4.1.3.1 Process Description
(WEF MOP 8, 4th edition)

A summary description and process diagram are provided in Figure 4.3. The trickling filter-solids contact (TF/SC) process typically includes a fixed film reactor that has low to moderate organic loads followed by a small contact channel. The contact channel is typically 5 to 15% (10 to 90 minutes) the size that would normally be required in an aeration basin for activated sludge alone. The aerated tank is intended to provide contact of the trickling filter effluent with the return biological solid to allow an adsorption process for enhanced removal of colloidal fractions not easily removed by settling in the final clarifier. By increasing the size of the contact tank additional BOD_5 removal can be obtained. By combining the fixed film reactor with a larger contact channel, the fixed film reactor size may be reduced by 10 to 30% of that normally required if treatment had been accomplished with a TF alone.

Benefits of the TF/SC process stem from reduced power requirements for the activated-sludge portion of the plant, because of the ability of the TF to remove most or all of the soluble BOD.

Conventional TF/SC does not include sludge re-aeration. However, when both solids contact and re-aeration are used, the acronym TF/SCR is used to describe the mode. When solids contact is eliminated and solids re-aeration is the only type of suspended-growth process used, then the acronym TF/SR describes the mode of TF/SC.

Under current practice, distinctions are made among TF applications based on the treatment provided rather than the hydraulic rate or organic loading of the application. This approach more accurately identifies the purpose of the TF operation. Hence, the general types of TFs are:

- Type 1: Roughing filters that provide approximately 50 to 75% $SBOD_5$ removal and 30 to 45% BOD_5 oxidation, followed by a second stage of treatment;
- Type 2: Complete treatment filters that provide the required settled effluent BOD5 and TSS;
- Type 3: Combined BOD5 removal and nitrogen removal filters that provide the required settled effluent quality for BOD5, TSS, and ammonium-nitrogen; and
- Type 4: Tertiary nitrifying filters that provide required effluent ammonium-nitrogen in a tertiary mode receiving a clarified secondary influent.

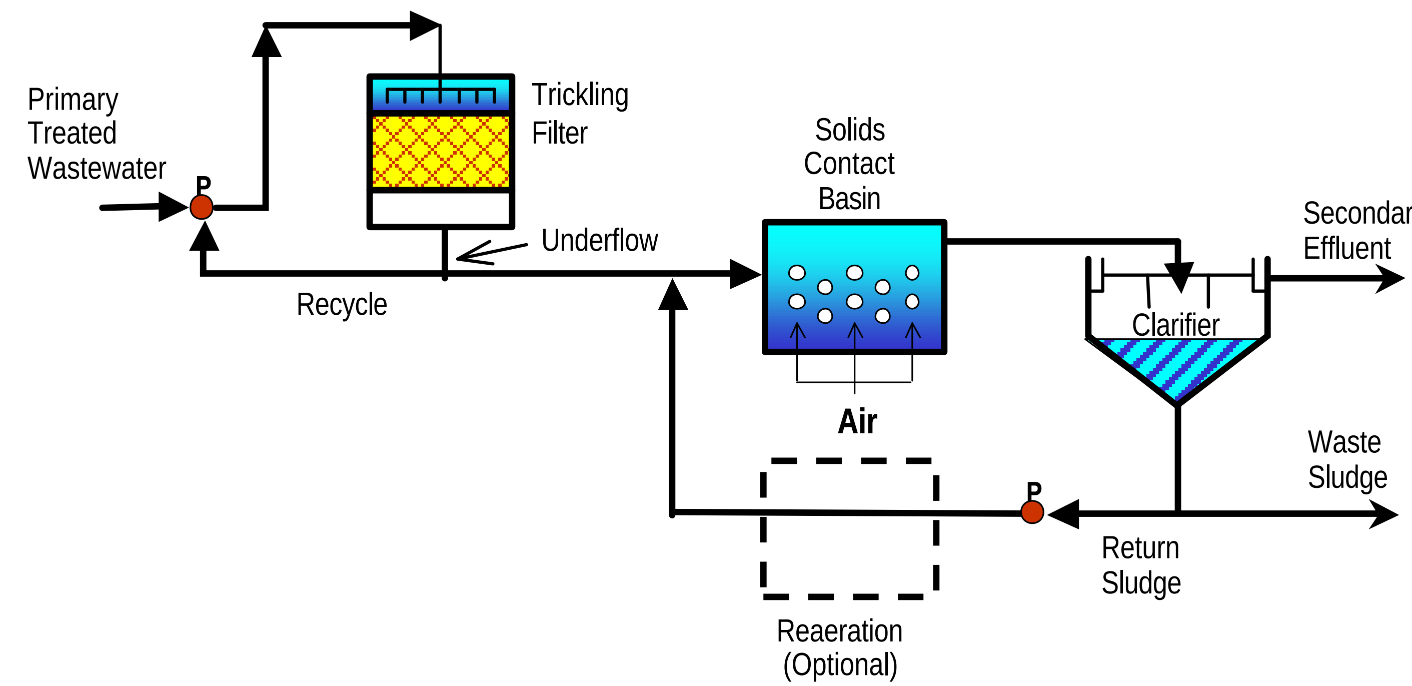
The TF/SC is the second type, unless the TF is a multi-stage system and/or the SC tanks are large; then they are able to assume the third type.

Adequately sized final settling tanks complete with flocculating zones are required to achieve proper effluent levels of TSS and BOD_5. Application of modern and deeper clarifier designs with energy-dissipating, center-feed wells, baffled launders, and moderate overflow rates are important to achieving good effluent quality.

Process Description

(See also Trickling Filter - Standard Rate) The Trickling Filter/Solids Contact (TF/SC) process consists of a trickling filter, an aerobic solids contact tank with a short HRT, and a final clarifier. The biological solids formed on the trickling filter slough off and are concentrated through sludge recirculation in the contact tank. The contact tank improves the settling characteristics of biomass sloughed from the trickling filters by flocculation and provides additional oxidation of carbonaceous organic material.

Schematics



Design Criteria

Trickling filter loading: 0.6 – 3.2 kg BOD/m^3.d
Solids Contact HRT: 10 - 60 min
Solids Contact SRT: 0.3 - 2.0 d
Solids Contact MLSS: 1000 - 3000 mg/L
Clarifier peak overflow rate: 1.8 - 3.0 m/h
Power: 8 - 16 kW/ML

Advantages

- Proven Technology
- Simple operation
- Robust process resistant to toxic and hydraulic shocks
- Low energy requirement for aeration
- Biomass cannot be washed out by high peak flows
- Does not suffer from filamentous bacteria, sludge bulking, foaming
- Biomass has excellent settling qualities
- Smaller footprint than activated sludge

Disadvantages

- Possible psychoda (filterfly) nuisance
- Odour potential from trickling filter
- Can have High trickling filter profile (visual)

Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Comment

- Has been used as existing GVRD plants with good results.

Plant Footprint

May be configured to fit on Lions Gate site

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4.1.3.2 Proven Technology
(WEF MOP 8, 4th Edition)

TF/SC is an established technology. TF Technologies currently available can produce Advanced Wastewater Treatment (AWT) effluents of 10 mg/L BOD_5 and TSS or less and ammonium-nitrogen effluents of 1 mg/L or less. Trickling filters have historically been considered vulnerable to climatic changes because wastewater droplets must be exposed to large volumes of ambient-temperature air. However, proper engineering design can reduce temperature losses caused by wind and ventilation to less than 1.5°C. Improving dosing procedures and minimizing recirculation can also help control temperature loss.

Temperature effects on nitrifying trickling filters are now considered to be less significant than those on activated sludge. Earlier observations of poor effluent quality in winter were caused by a combination of shallow filters with high surface area, low freeboard, and high recirculation ratios that caused excessive heat losses. Other conditions contributing to poor performance included poor clarifier designs and filter dosing procedures that caused excess solids accumulations.

Trickling filters are no longer viewed only as a process to produce secondary treatment effluent. The TF process now used for AWT produces low residual BOD_5, TSS, and ammonium-nitrogen. Replacing existing TFs is often more expensive than updating and expanding existing units using known process technology such as the addition of short-term aeration or the solids-contact process.

In applications where more stringent effluent quality standards have exceeded the capability of existing TF designs, expanding TF capabilities often meets the requirements. Based on recent experiences, the full potential of the TF is only now being realized. The improved treatment capabilities of new and modified facilities, along with inherent ease of operation and low power use, have resulted in continued use of TFs.

4.1.3.3 Discharge Requirements/Effluent Quality

The TF/SC process will easily meet the LWMP requirement for treated effluents to the environment of 45 mg/L BOD_5 and 45 mg/L TSS maximum allowable discharge. Should it be required, the TF reactors can be increased in height to provide ammonia removal through nitrification, but at increased cost.

4.1.3.4 Reliability
(WEF MOP 8, 4th Edition)

Successful conventional secondary and AWT applications are achievable with TFs but require a better understanding of TF operation and required appurtenances. If proper design procedures are used, TF performance equaling that of suspended-growth systems can be achieved:

- Trickling filters can produce effluent qualities of <10 mg/L BOD_5 and TSS;
- The effluent can be comparable to activated-sludge effluent;
- Trickling filters rapidly reduce soluble BOD_5 in applied wastewater;
- Temperature loss is less than 1.5°C in cold climates;
- Trickling filters are efficient nitrification units and effluents of <1.0 mg/L ammonium-nitrogen can be produced;
- Natural ventilation is inadequate for optimizing performance and power ventilation should be used;
- For rotating arm applicators, trickling filters should be dosed every 10 to 60 seconds, but routine flushing at 10 to 30 minutes/dose is also needed to enhance performance; alternatively, solid set and pumped application has wider flushing capability;
- Recirculation is typically beneficial for optimum performance, especially if the hydraulic loading rate is low;
- Power consumption is typically 25% less than activated-sludge treatment;
- Trickling filter sloughing cycles are harmful to filter performance and can be avoided by daily flushing; and
- Less land area is required for TFs than for activated-sludge treatment.

4.1.3.5 Site Suitability

The TF/SC facility is suited to either the Iona Island or the Lions Gate sites. For Iona Island, the site is of sufficient size to easily accommodate the TF/SC. For the Lions Gate site the TF/SC process could be added as a downstream process to the primary clarifiers or could convert the existing primary tanks to solids contact tanks and construct the TF over the tanks as was comparatively done in North West Langley WWTP. A smaller primary treatment (Salsnes filter or similar screening process) could be used in lieu of primary treatment as proposed for North West Langley WWTP. The primary treatment would be required to achieve an equivalent primary treatment to ensure all flows above 2 x ADWF were treated to at least primary if not treated in the TF/SC treatment train.

4.1.4 Rotating Biological Contactor

4.1.4.1 Process Description

The Rotating Biological Contactor (RBC) consists of a series of closely spaced circular disks made of polystyrene or polyvinyl chloride fixed to a central horizontal shaft. The disks are submerged in water and rotation of the disks takes place slowly. A process schematic and summarized technical facts are presented in Figure 4.4.

Biological growth become attached to the surfaces of the disks and during operation form a slime layer over the entire wetted surface. As the disc rotates the biomass alternately contacts organic material in the wastewater and the oxygen in the atmosphere and is thus maintained in an aerobic condition. Shearing forces are created as the disc rotates and, as a result, excess solids are sloughed off. These are kept in suspension to be carried from the unit to a clarifier.

The shaft to which circular disks are fixed is usually 3-7 m long and supported on bearings in semicircular steel, glass reinforced plastic or concrete tank. About 40% of the contactor usually lies below the surface of the effluent to be treated.

4.1.4.2 Proven Technology

RBC has been used in the wastewater treatment market for over 30 years. It was first installed in West Germany and was later introduced to the United States and Canada. 70% of the RBC systems installed are used for carbonaceous BOD removal only in the North America.

4.1.4.3 Discharge Requirement/Effluent Quality

RBC systems can be designed to provide secondary or advanced levels of treatment. Effluent BOD_5 characteristics for secondary treatment are comparable to well-operated activated sludge processes. The removal efficiencies of BOD_5 and TSS are both greater than 90% with the treatment of RBC. The process should comfortably meet:

- The 45/45 mg/l of BOD/TSS secondary effluent standard; and
- MBAS removal to <2 mg/l.

4.1.4.4 Reliability

RBCs are generally quite reliable when properly designed. This is because of the large amount of biological mass present (low F/M). The large biomass concentration also allows the system to withstand hydraulic and organic surges more effectively. The effect of staging in this plug-flow system eliminates short-circuiting and dampens shock loadings.

Nevertheless, early RBC units had operating problems such as shaft failures, media breakage and bearing failures. Units with increased submergence have been developed recently to reduce shaft and bearing loads and to improve equipment reliability.

FIGURE 4.4
ROTATING BIOLOGICAL CONTACTOR

Process Description

The rotating biological contactor consists of a series of closely-spaced circular disks of polystyrene or polyvinyl chloride (PVC). Bacteria become attached to the surface of the disks and eventually form a slime layer over the entire wetted surface area of the disks. The rotation of the disks alternately contacts the biomass with the organic material in the wastewater and then with atmospheric oxygen. The RBC can be used for secondary treatment, combined nitrification or as a separate nitrification component.

Design Criteria

Hydraulic Loading: 0.04 - 0.16 $m^3/m^2/d$
BOD Loading: 0.001 - 0.017 $kg/m^2/d$
Peripheral speed: < 20m/min.

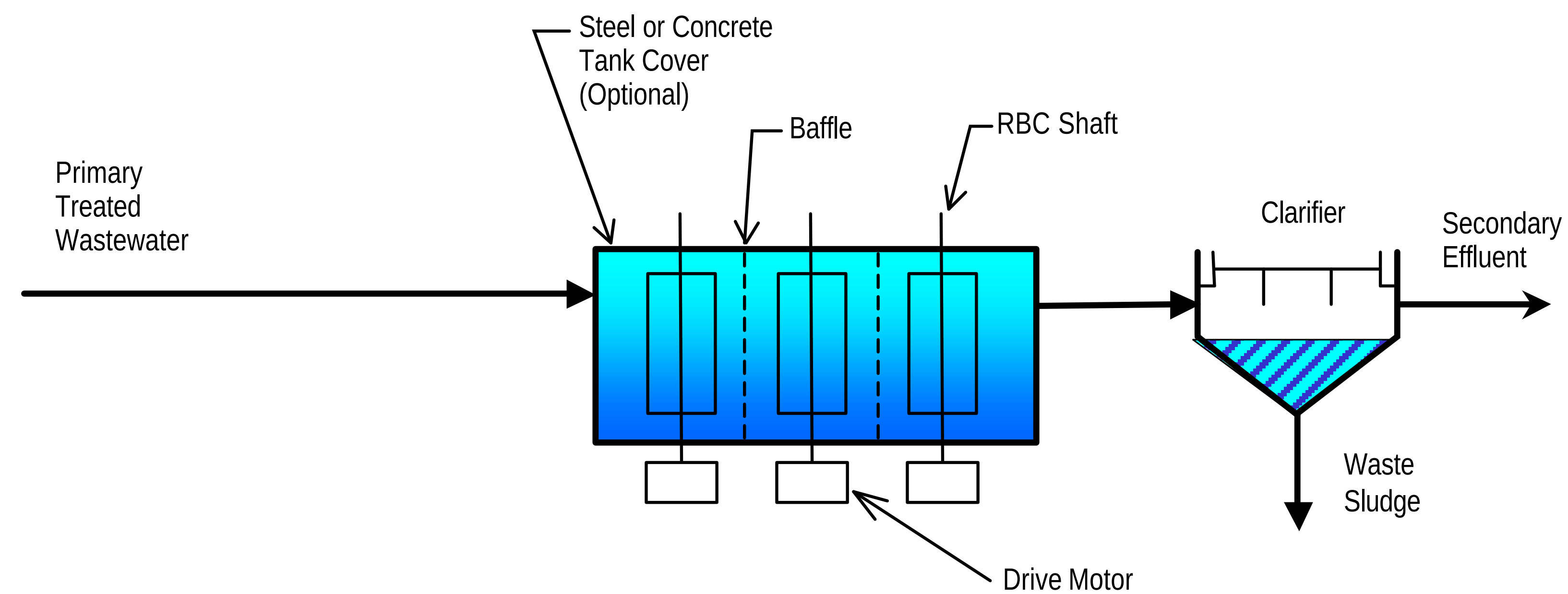
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Cannot fit on Lions Gate site

Schematics



Advantages

- Staging in the system eliminates short circuiting and dampens shock loadings.
- Well proven (mainly small facilities)
- Simple, robust process is resistant to hydraulic and toxic shocks
- Low energy for aeration if mechanical drives are used

Disadvantages

- Overloaded system results in low DO, H_2S odours, and poor first-stage removals.
- Large space requirement compared to other secondary processes

Comment

- Technology not yet proven for large-scale system like IIWWTP
- Large space required - not suitable for LGWWTP

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4.1.4.5 Site Suitability

Existing primary facilities at the treatment plants will continue to be used to treat raw wastewater before secondary treatment by the RBC. Extensive space is required for the RBC when compare to the conventional activated sludge treatment system (such as Trickling Filter system or BAF), which makes the RBC system unsuitable for LGWWTP. Also, the RBC applications have mainly been used for small communities. For a large-scale treatment plant like IIWWTP, the applicability of the RBC system is yet to be proven.

4.1.5 Biological Aerated Filter

4.1.5.1 Process Description

The description below has been extracted from Water Environment Federation Manual of Practice No. 8, 4th Edition.

There are many innovations in the processing of wastewater using submerged fixed media. These systems can be sorted into two basic categories:

1. Fixed film elements submerged in mixed liquor where there is sludge return from the secondary clarifier. These elements may be suspended in the mixed liquor (for example, Captor, KMT, and Linpor-C) or fixed (for example, Ringlace, submerged RBCs, Bio 2, and Sludge). The fixed film may or may not play the dominant role in biological treatment, depending on the design.

2. Fixed film elements and attached biomass are the primary mechanisms of the treatment process. Liquid may be recycled, but clarified sludge is not. These processes may use floating (Biostyr), submerged bed (for example, BioCarbone, Biofor), or fluidized-bed (for example, Oxitron, Biolift) media.

These processes have been used for BOD_5 removal, nitrification, and denitrification of municipal and industrial wastewater. A general objective of these processes is to complete the biological treatment in less space, and these designs may or may not be less costly. Future improvements in these processes are to be expected as experience is gained. This section will review only the fixed film processes that do not recycle sludge. The first group of processes, which are a combination of the fixed and suspended-growth biology, are discussed in Section 4.4 Fixed Film/Suspended Growth.

There are several developed processes and many technology programs underway throughout the world that relate to enhancing the performance of submerged fixed film reactors. Of the processes only a few are currently in commercial use.

In the subsequent discussion, only a few of the processes are discussed in more detail. These processes are typically in commercial operations, and there is full-scale experience available. Because this is a technical area that is very dynamic, this discussion could become dated in a short period. The design engineer must review the most up-to-date information and plant experiences before proceeding to use these types of processes.

4.1.5.2 Submerged Fixed Bed Reactors

The biological aerated filter (BAF) is similar to the design of high-rate sand filters except that air is continually sparged into the lower regions of the filter and a relatively coarse medium is used. The wastewater is downflowed or upflowed, and the granular medium retains influent suspended solids and provides a surface area for biofilm development. The media and fixed growth thus providing for removal of both wastewater contaminants and clarification in one unit. The media size in the reaction zone is tailored to the specific application.

Like a rapid sand filter, the progressive fouling of the media is evidenced by an increase in the depth of water above the media (or increased backpressure) because of increased head loss. Backwashing of the filter removes the accumulated TSS near the top of the filter media and excess biofilm growth throughout the media depth. The backwashing schedule is programmed based on head loss and time schedules that ensure maximum surface area available during peak flow periods.

4.1.5.3 Fixed Bed Reactor—Downflow Mode

Since the early 1980s, more than 100 BioCarbone® plants (Figure 4.5.1.1) have been constructed throughout the world. The municipal plant size has varied from 2,000 m^3/d, more typically from 4,000 m^3/d, to 80,000 m^3/d at Sherbrooke, Quebec. The plants are capable of providing secondary treatment and advanced wastewater treatment, including biological nutrient removal.

The BOD_5 loading rate for nitrification of the prototype plant at Geneva, Switzerland, was 2.5 $kg/m^3 \cdot d$, or 8 to 10 times as high a rate as that typically used in dual-purpose TFs.

4.1.5.4 Fixed Bed Reactor—Upflow Mode

The upflow mode of operation through a granular bed (Biofor®) has been used in more than 50 installations worldwide, treating wastewater from several millions of population equivalents. The Biofor® reactor is shown in Figure 4.5. The reactor may be used for COD and NOD removal and, with a carbon source, operates in a denitrification mode. The TSS are trapped in the media during normal operation and backwashed as required by increasing the hydraulic rate.

Media in the Biofor® reactor consist of expanded clay in the form of spherical grains with a diameter of 3.5 mm. Process water is introduced to the bed through a nozzle network in the reactor floor, and backwash water is introduced through the same network. Air is introduced to the bed through a separate network of diffusers located above the inlet nozzles. Influent flow must be fine screened to prevent blockage of the nozzles.

A 12-plant study of both upflow (Biofor®) and downflow (BioCarbone) BAFs indicated that both the upflow and downflow modes of this type of subsided, packed granular bed reactor produce comparable results.

4.1.5.5 Floating Bed Aerated Filters

The use of low-density media is a more recent development. Processes use a floating bed of media to provide the biological surface area and simultaneously act as a filtration system. Owing to the media size, the clean surface area is high. While coarse-bubble aeration diffusers are used, the media encourage excellent contact of air, water, and biomass. Only the Biostyr® will be reviewed because of limited information on other designs.

The Biostyr® unit (Figure 4.5) is a reactor that is partially filled with small (2 to 6 mm) polystyrene beads. The beads, being lighter than wastewater, form a floating bed in the upper portion of the reactor, typically occupying 60 to 75% of the total volume with approximately 1.5 m free zone below the bed. The top of the bed is restrained by a ceiling fitted with filtration nozzles to remove treated wastewater. The clean surface area of spherical particles is 1,000 to 1,400 m^2/m^3. The air distribution uses a diffuser network located along the bottom of the reactor or an aeration grid inside the media.

Upflowing wastewater is simultaneously treated and filtered to remove solids. Excess solids are removed by countercurrent water (lowering the water level) and backflushing and air scouring. Wash water is discharged to clarification for further treatment.

As of 1993, the Biostyr® process has been widely introduced to the European wastewater market and available technical information is rapidly expanding.

4.1.5.6 Fluidized-Bed Reactors

Fluidized-bed processes are a type of mobile beds where there is a uniform expansion of the media because of upflowing liquid. The first studies using fluidized beds to treat wastewater date back to the 1940s in England. Researchers at Manhattan College; U.S. EPA in Cincinnati, Ohio; and Water Research Centre in Medmenbam, England, contributed to the development in the 1970s. There are currently more than 80 two-phase fluidized-bed reactors operating in the U.S. and Europe for industrial wastewater treatment. Two-phase (Oxitron and Anitron) designs are suggested as a competitive, cost-effective process for anaerobic, anoxic, and aerobic treatment.

FIGURE 4.5
BIOLOGICAL AERATED FILTER (BAF)

Process Description

Primary effluent is pumped upwards or downwards through a bioreactor containing fixed media on the surface of which biomass grows. Essentially the proprietary bioreactor is a submerged aerated fixed film reactor. Air is injected in the form of fine bubbles, 1-2 mm in diameter near the base of the media in co-current flow with the primary effluent inlet stream. The biomass utilizes the organics in the wastewater as food and converts them to CO_2, water and additional biomass. The media is approximately 3 to 4.0 metres deep, has a high specific surface area, high porosity and is manufactured from materials which are resistant to attrition (e.g. Biofor® media consists of an expanded clay material). Periodically the bio-filters are backwashed and simultaneously agitated by air scour to wash biosolids from the media. Filter effluent is stored to provide backwash water. The backwash cycle can be controlled by a timed cycle and or head loss measurements. Multiple cells are utilized and can be cycled in and out of service to ensure generation at optimum flow rates for biological growth through a range of plant flows and load conditions.

Design Criteria

Total organic loading: 6 kg CBOD/$m^3 \cdot d$
Hydraulic loading: 4.5 m/hr (108 $m^3/m^2 \cdot d$) - average daily flow
10.5 m/hr (254 $m^3/m^2 \cdot d$) - peak flow
Media bed depth: 4 m
Overlying liquid depth: 1.5 m
Hydraulic retention time: 0.5 - 2.0 hours
Media bed backwash: 0.2 - 0.6 hr / 1 - 2 days
Backwash storage: 10% area of BAF reactor
Backwash equalization: 20% area of BAF reactor
Air requirement: 1.2 kg O_2 / kg BOD_5 removed

Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Can be accommodated on the Iona and Lions Gate sites

Schematics

Figure 4.5.1.1
The BioCarbone downflow biological process

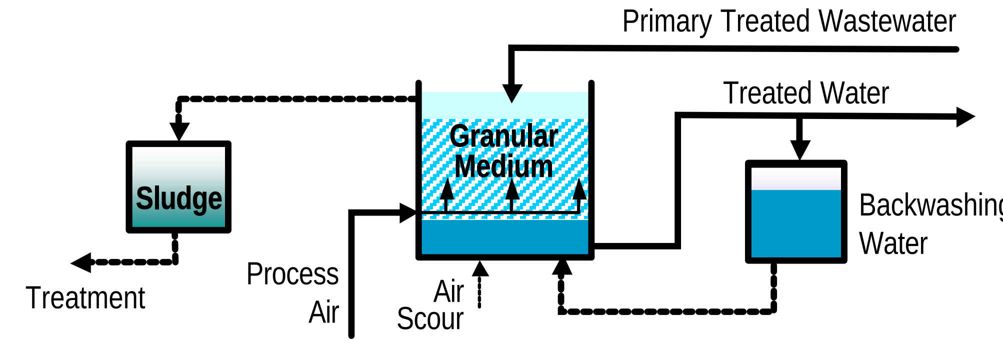


Figure 4.5.1.2
The Biofor® upflow biological reactor

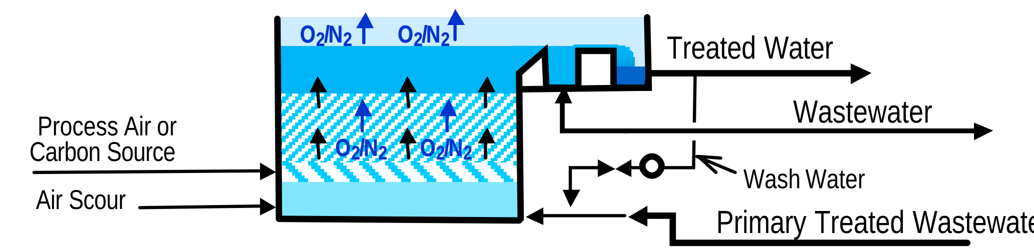
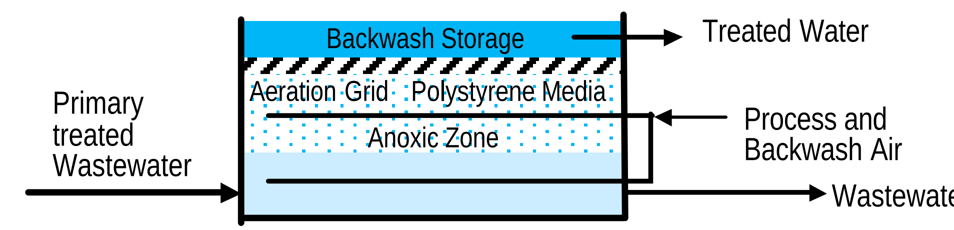


Figure 4.5.1.5
The BioStyr arrangement for nitrification-denitrification



Advantages

- No Final Clarifiers Required
- Smaller Footprint (~1/3) of CAS
- Excellent BOD and TSS Removal
- Easy to Upgrade for Nitrification / Denitrification (Add on BAF Cells)
- Odour Minimized
- Automated Operation
- Biosolids Amenable to Anaerobic Sludge Stabilization
- Sludge Quality about the same as CAS
- Can Easily be applied after CEP - Reduce Overall Capital Cost
- LWMP Effluent Goals Met - BOD, TSS
- Toxicity Reduced to LC_{50} of 100% Effluent

Disadvantages

- Limited Application in Canada
- Proprietary Technology
- High Energy Requirement (~40% more than CAS) - Aeration Pumps
- Effluent Storage and Pumping Required
- Backwash Cycle Load to the Primary Clarifiers
- Capital Cost Higher than CAS and TFSC ≈ 25%
- Need to Design for Peak Day Flows
- Sensitive to High Influent Loads
- High Backwash Flow (up to 20% of Influent Flow)
- Different Process than Annacis / Lulu

Comment

- Excellent candidate for application at Lions Gate or other sites with limited area. Probably not an economically competitive alternative for Iona Island.
- Currently being designed for use in conjunction with CEP at West Windsor Ontario

Despite the advantages of two-phase fluidized reactor beds, there have been problems in both scale-up and operation. These problems include control of bed expansion, limiting biofilm thickness, influent distribution, and oxygen saturation. Because these processes are still evolving, cited deficiencies are being or have been addressed and a current status should be determined by the design engineer. While investment costs may be 50% of activated sludge, operating costs are higher.

Three-phase fluidized beds are being evaluated as an alternative design to eliminate problems developed in two-phase units. These designs use simultaneous injection of wastewater and air (oxygen) through inlet nozzles. Current information is from pilot units, and reports indicate that bed expansion and good distribution of flow are still a problem.

The Biolift® process is a re-circulating media type being evaluated at Maxeville, France, at an operating scale of 2,400 m^3/d.

4.1.5.7 Comparison Of Treatment Technologies

The use of high-rate smaller volume biological reactors is being rapidly advanced. There is a wide range of reactor designs that may be fully fixed growth or a hybrid mixture of fixed and suspended growth. As would be anticipated with higher rate systems, effluent quality as defined by COD, BOD_5, and ammonium-nitrogen is less stable and daily variations can be appreciable. On the other hand, space requirements and perhaps costs are lower. These systems may be the only option for land-limited wastewater plants that need to expand from secondary to advanced levels of treatment. The greater susceptibility to peak loads needs to be addressed in the design.

Emerging process technology is rapidly advancing and information can become dated. It will be necessary for the engineer to review the most recent experiences and be sure that they are applicable to the site-specific design needs.

Because of readily available information and a proven track record, the Biofor process has been chosen as an example of the type of process which could be utilized. The process ultimately chosen should be selected based on a full evaluation of the available candidates at the time.

4.1.5.8 Proven Technology

The family of biological aerated filter processes has been used for more than 10 years and have in excess of 200 installations around the world with at least 5 in Canada. Of the plants treating municipal wastewater for BOD removal the capacities vary up to 265 ML/d PWWF. Reference sites are as follows:

Worldwide there are 60 Biofor plants with 9 in the range 100 to 410 ML/d PWWF. Two of these are in Canada (City of Quebec and Thunder Bay) One of the 9 is a BOD removal plant; the remainder are used for tertiary nitrification. There are a number of smaller BOD removal plants. In Canada a total of 5 plants have been built at Thunder Bay, Royal Polymers, Town of Canmore, City of Chateauguay, and the City of Quebec.

Worldwide there are 68 Biostyr plants with 12 in the range 100 ML/d to 1,700 ML/d PWWF. Of these, 2 are in the United States of America (Denver and Onondaga). One is a BOD removal plant with the balance being used for tertiary nitrification. 8 plants have been designed for BOD removal. 6 plants have been built in North America.

4.1.5.9 Discharge Requirements

The effluent quality should comfortably meet the required 45/45 secondary standard. Should ammonia conversion to nitrate be required this could be achieved by adding on additional units. De-nitrification could be achieved in a similar manner.

4.1.5.10 Reliability

Problems, which have occurred during early full-scale implementation, have been addressed progressively over time. Careful selection of an appropriate proprietary configuration should provide the necessary level of assurance. Changes in influent sewage characteristics should be accommodated by the adaptation of the biomass over a short time. Daily fluctuations in flow and load would be accommodated by varying the number of units on line at any time. Control system disruption would adversely affect the operation of the plant.

4.1.5.11 Site Suitability

These processes are noted for the small footprint which they require. They are therefore particularly suited to the Lions Gate WWTP site.

4.2 SUSPENDED GROWTH

4.2.1 Conventional Activated Sludge (CAS)

4.2.1.1 Process Description

Conventional activated sludge (CAS) with aerated bioreactors and secondary clarifiers can be applied as the main process to achieve secondary treatment effluent quality. A process schematic and technical information are summarized in Figure 4.6. Following primary clarifiers, the primary effluent is aerated in bioreactors with the presence of activated microorganisms (mixed liquor suspends solids, MLSS) for approximately 6 hours. The activated sludge microorganisms utilize organics that remain in the primary effluent as a food source and convert it to biomass, carbon dioxide and water, resulting in BOD_5 removal. Certain amount of nitrogen contents will be removed by biomass synthesis, and nitrification will convert ammonia into nitrate if the conditions allow. The MLSS solids remaining in the bioreactor effluent are settled out in the secondary clarifiers to achieve TSS removal.

Process Description

Following primary clarifiers, the primary effluent is aerated in bioreactors with the presence of activated microorganisms (mixed liquor suspends solids, MLSS) for approximately 6 hours. The activated sludge microorganisms utilize organics that remain in the primary effluent as a food source and convert it to biomass, carbon dioxide and water, resulting in BOD_5 removal. Certain amount of nitrogen contents will be removed by biomass synthesis. The MLSS solids remained in the bioreactor effluent are settled out in the secondary clarifiers to achieve TSS removal.

Approximately 50 - 75% of the MLSS is wasted. The remaining 25 - 50% of the MLSS is recycled as return activated sludge (RAS) to the bioreactor to seed the process. Compressed air is applied to the bioreactor to maintain the micro-organisms in an aerobic condition.

Design Criteria

F/M: 0.4 -1.0 kg BOD_5 / kg MLSS d
SRT: 3 - 6 days
MLSS: 2,500 mg/L
RAS rate: 25 - 50% Q
HRT: 6 hours
Tank depth: minimum 4.5 m
Secondary settling tank SOR: ~18 m^3/m^2 day
Solids loading rate: ~5.0 kg/m^2 day
Air requirements: 1.2 kg O_2 / kg BOD_5 removed
or 7.5 m^3 of air /day m^3 of bioreactor capacity

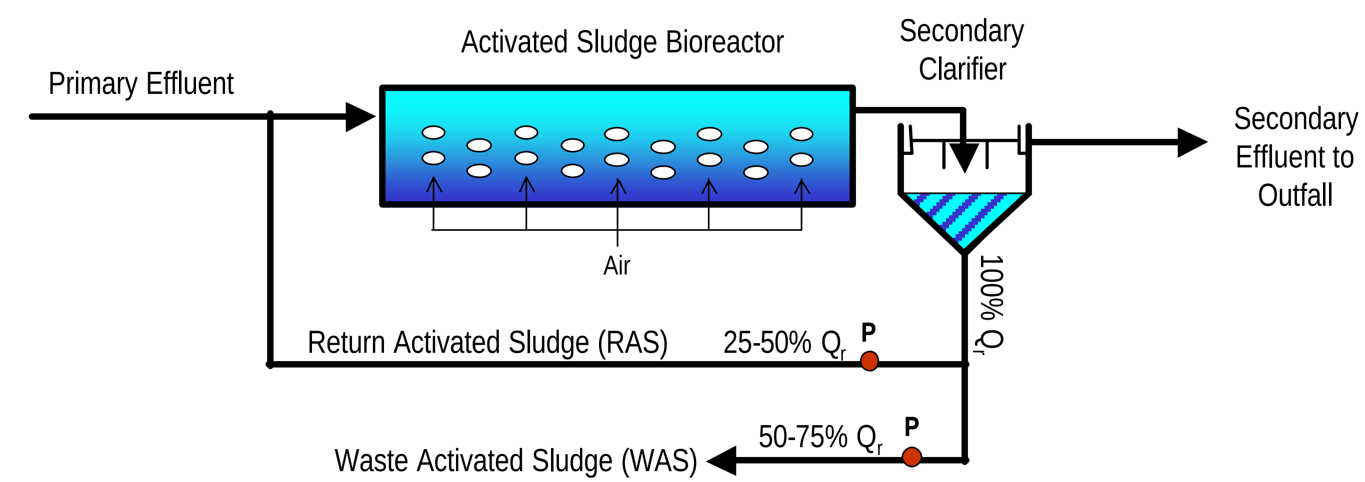
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	30
TSS mg/L	130	70	30

Plant Footprint

IIWWTP: Approximately 66,000 m^2 + primary and solids handling upgrade
LGWWTP: Approximately 16,000 m^2 + primary and solids handling upgrade

Schematics



Advantages

- Proven technology
- Less sludge than CEP
- LWMP effluent goals met - BOD, TSS
- Toxicity reduced to LC_{50} of 100% effluent
- Re-use entire process - secondary build-out
- Odour problem limited
- Good quality biosolids
- Operating costs lower than CEP

Disadvantages

- Significant plant footprint
- Additional staff required
- Land space limitation at LGWWTP
- Different process than AIWWTP and LIWWTP
- Capital costs higher than CEP

Comment:

- Could be a good option for IIWWTP, but poor choice for LGWWTP due to cramped site

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Approximately 50 to 75% of the MLSS is wasted (waste activated sludge, WAS, also referred as biosolids), thickened and directed to sludge stabilization (anaerobic digesters). The remainder 25 to 50% of the MLSS is recycled as return activated sludge (RAS) to the bioreactor to maintain the solids retention time (SRT) at about 3 - 6 days, and MLSS concentration at about 2,500 mg/L. However, the operational conditions can be adjusted to accommodate the flow and load variances in different seasons. Air supply is required in the bioreactors (usually compressed air using blowers) to provide the oxygen demand of microorganisms in an aerobic condition and prevent anaerobic fouling and odour potential.

The sludge production is expected to increase by 130 - 150% compared with the existing primary treatment process. The sludge handling capacity needs to be upgraded accordingly. The odour is generally not a problem in the CAS operation if the aerobic condition can be properly maintained. The level of operation and maintenance (O/M) in the CAS (e.g. WAS, RAS, SRT, MLSS, and DO controls) is similar to other biological processes. The plant has to be operated by certified staff.

4.2.1.2 Proven Technology

CAS has been widely used worldwide for more than 50 years. CAS is also one of the most common and cost-effective treatment processes for large-scale municipal wastewater application in North America. Design and operation experience have been developed over the years providing solid technical support for the application of such treatment. Modified activated sludge processes can be employed to achieve advanced removals such as nitrogen and phosphorus to meet stringent nutrient effluent quality requirements in the future. There are more than 50 CAS plants in Northwest America, including Bellingham Post Point WWTP (45 ML/d), Washington, and Bozeman WWTP, Montana (30 ML/d).

4.2.1.3 Discharge Requirements/Effluent Quality

The effluent quality of the CAS effluent is expected to achieve 30 mg/L of BOD and 30 mg/L of TSS, respectively. Effluent toxicity reduction is also possible due to additional BOD_5, NH_3-N and surfactant removals.

4.2.1.4 Reliability

The operation of the activated sludge process requires constant attention in order to maintain the biological culture in the process. Acclimation of the activated sludge system is also necessary to establish the optimum condition for BOD and TSS removal. System upsets may occur due to changes from influent shock loading, operational condition, and toxic substances. Deterioration of effluent quality is expected due to the loss of the biological microorganisms. Preventative measures must be taken to accommodate the upset factors. Otherwise, it usually takes several days or a week for recovery following upsets.

The secondary treatment units are designed to treat two times the ADWF flow, therefore, excess flow above the design value should be bypassed to prevent any system upset. The operational conditions, e.g. MLSS concentrations, RAS and SRT should be adjusted accordingly with the seasonal variance (e.g. dry weather and wet weather flows) to assure the system performance and stability.

4.2.1.5 Site Suitability

The CAS process requires significant land space for the construction of bioreactors and secondary clarifiers. The handling capacity of sludge thickening and digestion should be expanded accordingly for the increase of sludge production. For secondary build-out to treat 100% of the design flow by the CAS process, approximately 66,000 m^2 and 16,000 m^2 of footprint are required for IIWWTP and LGWWTP, respectively, plus solids handling upgrade. CAS is considered a good candidate for IIWWTP, but it appears that there is insufficient space at LGWWTP.

4.2.2 High Rate Activated Sludge (HRAS)

4.2.2.1 Process Description

High rate activated sludge (HRAS) process is modified from the conventional activated sludge (CAS) with higher MLSS concentrations (4,000 - 10,000 mg/L) and higher volumetric loadings. A process schematic and some technical information are summarized in Figure 4.7. Following primary clarifiers, primary effluent is treated in the bioreactors and aerated for approximately 3 - 4.5 hours, or less. The organic matter is utilized by the microorganisms grown in the bioreactors, and the solids and biomass are settled in the sequential secondary clarifiers. The bioreactor volume of HRAS can be sized about 30 to 50 % smaller than CAS at the same capacity, which requires less land space. The food to biomass ratio (F/M) in HRAS is higher than in CAS, and shorter SRT is maintained in HRAS.

Adequate aeration and mixing is very important to the HRAS operation, since high MLSS concentration is maintained in the bioreactors. The secondary clarifiers should also be designed to handle high solids loading. HRAS is often operated without primary clarifiers, therefore, if the existing primary settlers are retained in operation, the bioreactor condition should be properly designed to maintain high rate operation (e.g. F/M ratio, MLSS concentration and SRT etc.). It is also possible to convert the primary tanks into bioreactors if their structural conditions are suitable, however the main limitations are reactor depth and seismic standards. The HRAS sludge production will be about the same as the CAS process.

HIGH RATE ACTIVATED SLUDGE (HRAS)

FIGURE 4.7
HIGH RATE ACTIVATED SLUDGE (HRAS)

Process Description

This process is a modification of CAS. High MLSS concentrations are combined with high volumetric loadings. Mean cell residence time (MCRT or SRT) is longer and F/M ratio is higher. Removal Efficiency is not as good as CAS. Bioreactors for this process are much smaller than CAS (30% to 50% of CAS).

Adequate aeration and mixing is very important to the HRAS operation, since high MLSS concentration is maintained in the bioreactors. The secondary clarifiers should also be designed to handle high solids loading. HRAS is often operated without primary clarifiers, therefore, if the existing primary settlers are retained in operation, the bioreactor condition should be properly designed to maintain high rate operation (e.g. F/M ratio, MLSS concentration and SRT etc.). The HRAS sludge production will be about the same as the CAS process.

Design Criteria

F/M: 0.5 - 1.5 kg BOD_5 / kg MLSS d
SRT: 0.75 - 2 days
MLSS: 4,000 - 10,000 mg/L
RAS rate: 100 - 500% Q
HRT: 3 - 4.5 hours
Tank depth: 4.5 - 5.0 m
Secondary settling tank SOR: ~18 m^3/m^2 day
Solids loading rate: ~5.0 kg/m^2 day
Air requirements: 1.2 kg O_2 / kg BOD_5 removed
or 7.5 m^3 of air / day / m^3 of bioreactor capacity

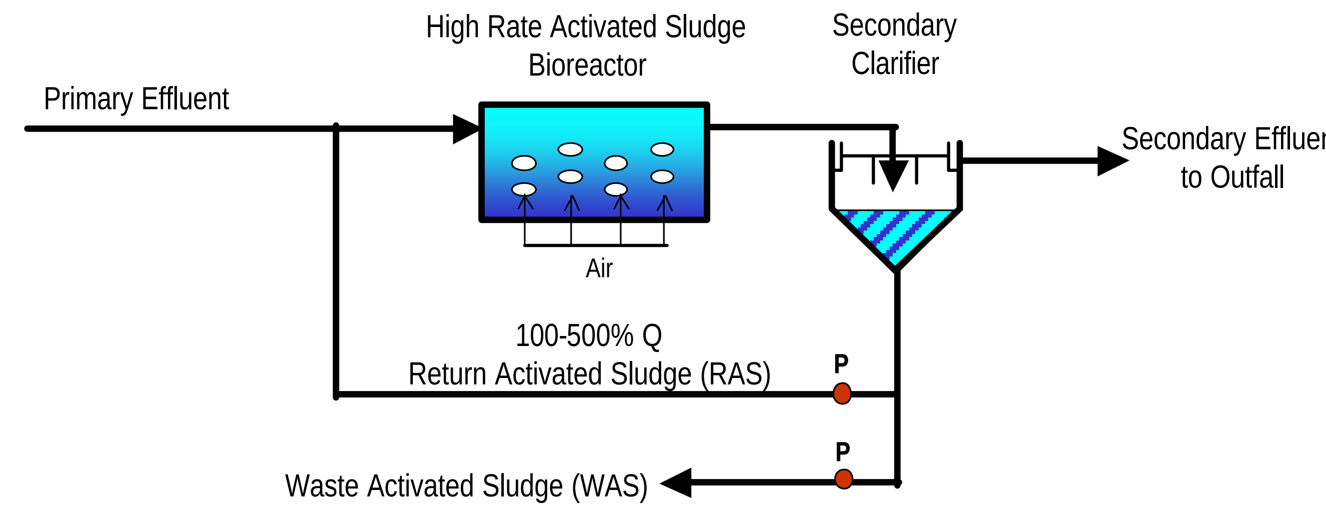
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately 51,000 m^2 + solids handling upgrade
LGWWTP: Approximately 12,000 m^2 + solids handling upgrade

Schematics



Advantages

- Proven technology e.g. New York City
- Interim LWMP effluent goals met - BOD, TSS
- Primary treatment is not required
- Odour control requirement limited
- Good quality biosolids
- Operating costs lower than CEP

Disadvantages

- Significant plant footprint
- Additional staff required
- Land space limitation at LGWWTP
- Different process than Annacis / Lulu
- Capital costs higher than CEP
- Slightly lower removal efficiency than CAS

Comment

- Could be a good option for IIWWTP, but probably poor choice for LGWWTP because of cramped site. Unless some of the primary clarifiers are no longer used in process and the HRAS can be built in existing clarifiers

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4.2.2.2 Proven Technology

HRAS has been widely used in municipal and industrial wastewater treatment as an alternative to CAS. Several installations in North America include Kalispell WWTP (27 ML/d), Montana, Twin Fall WWTP (30 ML/d), Idaho, Newton Creek (1200 ML/d), New York, and Western Branch WWTP, Maryland (75 ML/d). The operational requirement of HRAS is similar to the CAS process.

The removal efficiency of HRAS is usually about 10% lower than CAS, in terms of BOD_5 and TSS removal efficiencies. The effluent quality with HRAS is expected to achieve 45 mg/L of BOD_5 and 45 mg/L of TSS, respectively. Effluent toxicity reduction is possible to achieve 100% LC_{50}, by removals of additional BOD_5, NH_3-N, and potentially surfactants.

4.2.2.3 Reliability

HRAS requires similar attention as the CAS process to maintain the stability of the biological cultures for proper treatment. System upsets due to shock loading, toxic substances, and abnormal operational conditions are expected to be similar to the CAS process. Shorter acclimation time is anticipated in HRAS due to shorter SRT than in CAS. However, effluent quality deterioration may last for a couple days or a week.

The operational conditions, e.g. MLSS concentrations, RAS and SRT should be adjusted accordingly with the seasonal variance (e.g. dry weather and wet weather flows) to assure the system performance and stability. Excess flow above the design value (i.e. two times of ADWF) should be bypassed to prevent hydraulic shock load.

4.2.2.4 Site Suitability

Similar to CAS, HRAS requires significant capital investment of the bioreactor and secondary clarifiers, as well as footprint expansion. For the secondary build-out capacity, approximately 51,000 m^2 and 12,000 m^2 of footprint are required for IIWWTP and LGWWTP, respectively. The existing sedimentation tanks can be demolished and the space can be used for the bioreactor structures. The HRAS can be a good option for IIWWTP to achieve secondary treatment goals, however, the site space is limited at LGWWTP to implement the expansion.

4.2.3 Oxidation Ditch

4.2.3.1 Process Description

Oxidation ditch is a modification of the extended aeration activated sludge process that provides long solids retention time (SRT) and high mixed liquor suspended solids (MLSS) concentration. A conventional oxidation ditch consists of ring, oval or horseshoe-shaped channel bioreactors, with continuous flow, mechanical mixing and aeration. The BOD removal, nitrification and potentially denitrification can be achieved in the bioreactors, and solids and MLSS are settled in the following secondary clarifiers.

A process schematic and technical information are summarized in Figure 4.8. A common carousel type oxidation ditch is operated in which internal baffles are installed to enhance the plug-flow pattern. Vertical turbine aerators are used to provide aeration, mixing and drive the flow in circulation. Screened and degritted wastewater enters the oxidation ditch and circulates at about 0.25 – 0.35 m/s over the entire length of ditch. Such a velocity ensures contact between microorganisms and the incoming wastewater. The amount of oxygen required is estimated to be 1.1-1.5 kg O_2/kg BOD removed, depending on the maximum diurnal organic loading, MLSS concentration, and degree of treatment. The depth of channel is typically about 1 ~ 3 m, which is suitable for either turbines or mechanical aerators.

Solids from the oxidation ditch effluent are separated in a secondary clarifier where settled sludge is recycled (or returned activated sludge, RAS) back to the oxidation ditch at a ratio of 75 – 150% of influent flow. The MLSS concentration is maintained typically at 3,000 ~ 5,000 mg/L in the bioreactors. The food to microorganism (F/M) ratio is approximately 0.05 - 0.30 kg BOD_5 / kg MLVSS·d. The volumetric loading is usually designed to be about 0.1 - 0.5 kg BOD/m^3·d. The process has long hydraulic retention time (HRT) of 12 - 36 hours and long solids retention time (SRT) of 15 - 30 days compared to other activated sludge processes. Disinfection and reaeration may be necessary prior to final discharge of effluent. The oxidation ditch can be configured with alternate aerobic and anoxic zones to provide nitrification and denitrification, such as DO set point control and intermittent aeration.

Oxidation ditch is economical for small plants. It produces high quality effluent and is capable of treating shock loads. The waste sludge generated is well stabilized and has lower sludge yield than the conventional activated sludge processes. Recent developments have shown that it would be possible to operate an oxidation channel with a depth of up to 8 m (deep oxidation ditch). As a result the footprint would be significantly reduced.

Process Description

The oxidation ditch is a variant of the extended aeration activated sludge process. It consists of a mixed and aerated ring- or oval-shaped channel. Screened wastewater enters the ditch and circulates at about 0.25 to 0.35 m/s. Oxidation ditches typically have long hydraulic detention and solids retention times. Secondary sedimentation tanks with return activated sludge pumping are required.

Oxidation ditch is economical for small plants. It produces high quality effluent and is capable of treating shock loads. The waste sludge generated is well stabilized and has low sludge yield than the conventional activated sludge processes. The oxidation ditch process is commonly designed with a maximum capacity of 45,000 m^3/day per module, including the bioreactor and the associated secondary clarifiers.

Design Criteria

SRT: 15 - 30 days
F/M: 0.05 - 0.30 kg BOD/kg MLVSS.d
Volumetric Loading: 0.1 - 0.5 kg BOD/m^3/d
MLSS: 3000 - 5000 mg/L
RAS: 75 - 150% of influent
Water depth: 1.0 - 3.0 m
HRT: 12 - 36 hours

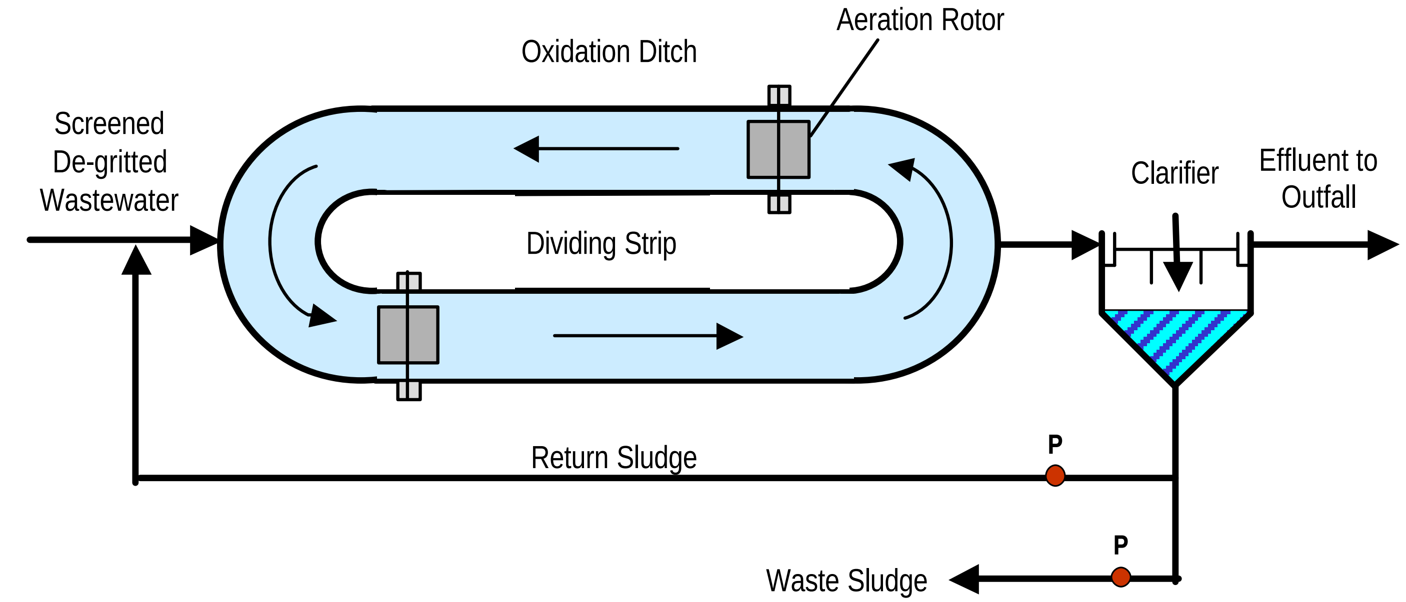
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately 460,000 m^2 + additional sludge handling capacity
LGWWTP: Approximately 110,000 m^2 + additional sludge handling capacity

Schematics



Advantages

- Highly reliable process
- Simple operation
- Capable of treating shock/toxic loads without compromising effluent quality
- Economical process for small plants
- Adaptable to nutrient removal
- High-quality effluent possible
- Well-stabilized waste sludge and low sludge yield

Disadvantages

- Long detention time
- Large structure, greater area required
- Some oxidation ditch process modifications are proprietary and license fees may be required
- Subject to foaming, filamentous infestations, sludge bulking

Comment

- Could be an economic option for IIWWTP, but the space requirement will limit its application at LGWWTP.

4.2.3.2 Proven Technology

Oxidation ditch originated in Netherlands in the 1950s and the first full-scale oxidation ditch in U.S. was commissioned in 1963 at Beaverton, Oregon. The process is widely adopted and now has more than 9,000 installations in the U.S. It has proven to be an effective secondary treatment technology producing high quality effluent. Total nitrogen removal is also possible with proper process control.

The oxidation ditch process requires a larger footprint than the conventional activated sludge process. However, for an equivalent level of treatment, lower capital and O/M costs are to be expected. Recently, several local treatment plants have adopted the oxidation ditch for their capacity expansions. These include the District of Campbell River (1996), District of North Cowichan (1996), and Capital Regional District (2000).

4.2.3.3 Discharge Requirement/Effluent Quality

The effluent quality is expected to achieve the 45 mg/L of BOD_5 and 45 mg/L of TSS standards, with BOD_5 and TSS removal efficiencies greater than 90%. The total N removal efficiency is expected to be over 50% and the ammonia-nitrogen is expected to be below 15 mg/L as N in the effluent.

4.2.3.4 Reliability

Oxidation ditch is capable of treating influent shock loads within a certain design range. The process is considered reliable for achieving high quality of effluent and adaptable to nutrient removal. Full-scale experiences have demonstrated that it has simple operation and requires lower operation and maintenance costs than other secondary treatment processes. However, the process is subject to foaming, filamentous infestations and sludge bulking, which may result in system upsets. Preventative controls must be prepared to maintain a stable system.

4.2.3.5 Site Suitability

The oxidation ditch process is commonly designed with a maximum capacity of 45,000 m^3/day per module, including the bioreactor and the associated secondary clarifiers. For the design secondary build-out capacity, IIWWTP needs 23 modules and LGWWTP needs 6 modules. The total footprints required for the oxidation ditch option are approximately 460,000 m^2 at IIWWTP and 110,000 m^2 at LGWWTP, plus additional sludge handling capacity. The capital investment of the aeration system is considered lower than the conventional activated sludge processes.

Oxidation ditch plants do not normally include primary clarifiers. At an existing plant, the modification of the primary clarifiers, subject to their effective depth and structural condition, may therefore be an option. Alternatively, it may be economically viable to demolish the existing pre-aeration and primary sedimentation tanks and to build oxidation ditches and secondary clarifiers on the same site. The oxidation ditch process can be considered for IIWWTP, however due to space constraints, it may not be a feasible option at LGWWTP.

4.2.4 High Purity Oxygen Activated Sludge

4.2.4.1 Process Description

Since 1970 there have been nearly 300 high purity oxygen (HPO) activated sludge systems constructed worldwide both in industrial and municipal applications.

The HPO activated sludge process was developed and commercialized by Union Carbide (Linde Division). The initial full-scale pilot program sponsored by the Federal Water Quality Administration (predecessor to the Environmental Protection Agency), EPA, was performed at a municipal plant in Batavia, NY from late 1968 and completed in 1970. In 1970, Union Carbide began commercialization of this process which they called the UNOX® system. A simplified schematic of the system is shown in Figure 4.9.

After more than a decade of successful commercialization of the HPO system the technology was acquired by Latepro in 1981. Currently of the more than 110 operating HPO systems in municipal service in North America, approximately 1 in every 6 were designed for biological nutrient removal. Plant sizes vary up to 750 ML/d. The system is similar to a conventional surface aeration activated sludge system except that the bioreactor tanks are covered and 85% pure oxygen is introduced to the first compartment.

The oxygen partial pressure in the headspace may range from 40 to 60 percent in the first stage to 20 percent in the last stage. At high oxygen partial pressure, higher volumetric oxygen transfer rates are possible so that pure oxygen systems can have a higher MLSS concentration and operate at a shorter sludge age and higher volumetric organic loadings than conventional processes. The rate of oxygen absorption is about 2 to 3 times greater than that of conventional aeration systems.

Onsite oxygen general equipment is needed to provide the pure oxygen gas for the process, making the process operation more complex than conventional activated-sludge processes. Nitrification ability is limited with the high-purity oxygen processes due to the accumulation of carbon dioxide in the gas headspace, which causes low pH in the mixed liquor (less than 6.5). Major advantages for pure oxygen systems are the reduced space requirement and greatly reduced quantities of off-gas if odour control and VOC control are required. Oxygen is either generated on site by a Pressure Swing Absorption for smaller plants or cryogenic oxygen system or is imported from a commercial supplier as liquid oxygen (LOX).

FIGURE 4.9
HIGH PURITY OXYGEN ACTIVATED SLUDGE (HPO)

Process Description

Primary effluent from the existing primary settling tanks at Iona and Lions Gate will be discharged to a high purity oxygen activated sludge bioreactor. Similar to the conventional activated sludge system, the mixed population of preconditioned organisms (activated sludge) will utilize organics in the wastewater as a food source and will convert them to activated sludge (AS) cells, CO_2 and water. Suspended solids in the influent will either be degraded biologically or entrapped in the AS and settled out in the final clarifiers along with the biosolids. A portion of the sludge will be returned to the inlet of the bioreactor to seed the process. High purity oxygen - 95% O_2 - will be added to the bioreactor enabling the process to be operated at high mixed liquor suspended solids levels of 8,000 to 10,000 M/L; therefore, the bioreactor will be much smaller than a conventional activated sludge bioreactor (50% of CAS volume). High purity oxygen will either be generated on site or liquid oxygen will be purchased from commercial suppliers.

Design Criteria

F/M: 0.4 kg BOD_5 / kg MLSS.d
Sludge age: 3 - 6 days
MLSS: 8,000 mg/L
RAS rate: 25 - 50% Q.
HRT: 3 hours
Tank Depth: 4.5 m

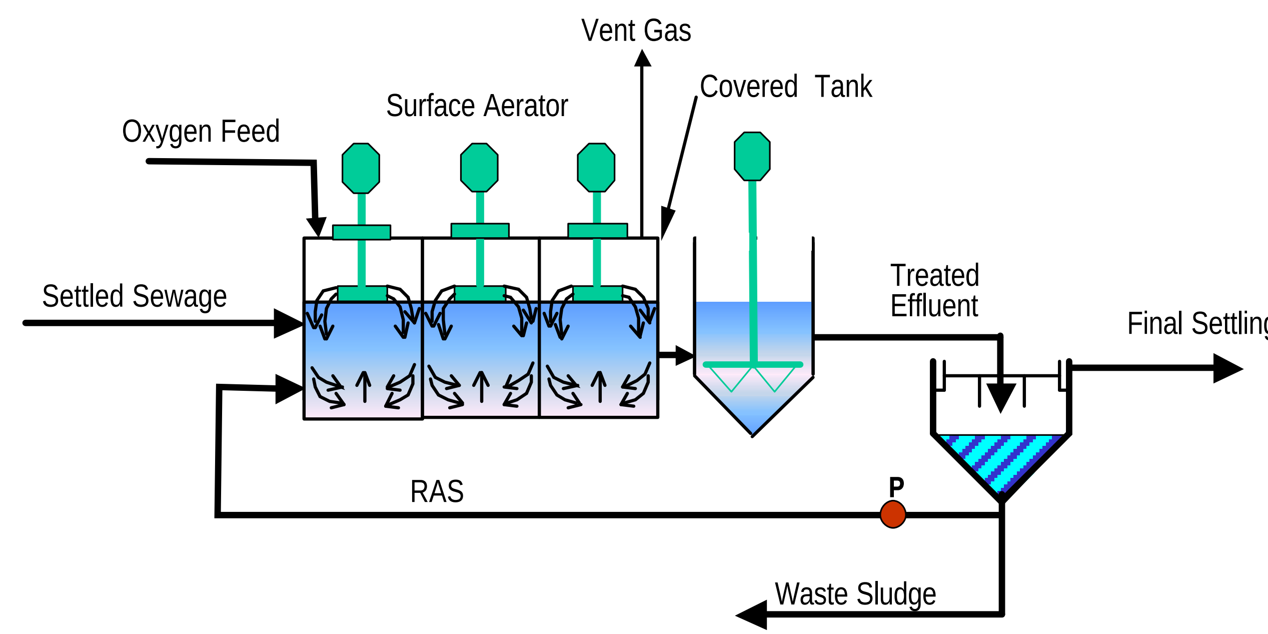
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

May fit on Lions Gate site.

Schematics



Advantages

- Proven Technology - Winnipeg, Manitoba; Calgary, Alberta
- LWMP Goals Met - BOD, TSS
- Smaller Plant Footprint than CAS
- Less gas/odour emission

Disadvantages

- Higher Operating Costs than (CAS) and TFSC
- Higher Capital Costs than (CAS) and TFSC
- No Advantage for Application at Iona
- Different Process than Annacis Island and Lulu Island
- More Complex Process and More Difficult to Operate than CAS
- Depressed pH due to CO_2 accumulation, may need chemical dosing

Comment

- Could be a candidate for Lions Gate because of smaller footprint and aesthetics of bioreactor
- Not a candidate for Iona (cost).

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4.2.4.2 Proven Technology

The technology has been utilized since 1970 and many plants of all sizes have been built in North America. The technology has therefore been well established on a commercial scale.

4.2.4.3 Discharge Requirements/Effluent Quality

The technology should comfortably meet the 45/45 BOD/TSS secondary discharge standard.

4.2.4.4 Reliability

Because the process tends to depress the pH, unless control is exercised, a pH drop, which could result in reduced effluent quality, may occur. In all other respects, the process is as robust as conventional activated sludge (CAS).

4.2.4.5 Site Suitability

Because of the oxygen transfer rate, which is 2 to 3 times that of CAS, a bioreactor approximately half the size of a CAS bioreactor can be utilized. This saving in area is somewhat offset by the requirement for a larger clarifier and the oxygen production facility. However, a smaller overall foot print results. The higher cost of construction and operation is not justified at Iona Island but HPO could be applicable at Lions Gate WWTP because of the smaller footprint.

4.2.5 Multi Anoxic Step Feed (MASF)

4.2.5.1 Process Description

Multi anoxic step feed is a modification of the separate stage activated sludge process to achieve organic and nitrogen removal. A typical process schematic and technical information is summarized in Figure 4.10. The process consists of anoxic and aerobic zones in each module of a multiple-module setup (typically four modules in series), followed by the secondary clarifiers.

The aerobic zones are operated to achieve carbonaceous BOD removal and nitrification (converting ammonia-nitrogen to nitrate). The primary effluent is introduced at the anoxic zone of each module to maximize the use of organic substrate for denitrification (converting nitrate to nitrogen gas), minimize the aeration demand (denitrification credit), and reduce the overall bioreactor volume. Chemical supplement of readily biodegradable carbon source can also be optimized with this multiple feed setup to improve the denitrification rate, if needed.

FIGURE 4.10
MULTI ANOXIC STEP FEED (MASF)

Process Description

Multi anoxic step feed is a modification of the separate stage activated sludge process to achieve organic and nitrogen removal. The process consists of anoxic and aerobic zones in each module of a multiple-module setup (typically four modules in series), followed by the secondary clarifiers. The aerobic zones are operated to achieve carbonaceous BOD removal and nitrification (converting ammonia-nitrogen to nitrate). The primary effluent is introduced at the anoxic zone of each module to maximize the use of organic substrate for denitrification (converting nitrate to nitrogen gas), minimize the aeration demand (denitrification credit), and reduce the overall bioreactor volume. Chemical supplement of readily biodegradable carbon source can also be optimized with this multiple feed setup to improve the denitrification rate, if needed. The sludge production is expected to be less than the conventional activated sludge process because of longer SRT.

Design Criteria

F/M: varied
SRT: 6 - 18 days
MLSS: 2,500 - 5,000 mg/L
RAS rate: 100 - 500% Q
HRT: 3 - 4.5 hours
Tank depth: 4.5 - 5.0 m
Secondary settling tank SOR: ~18 m^3/m^2 day
Solids loading rate: ~5.0 kg/m^2 day

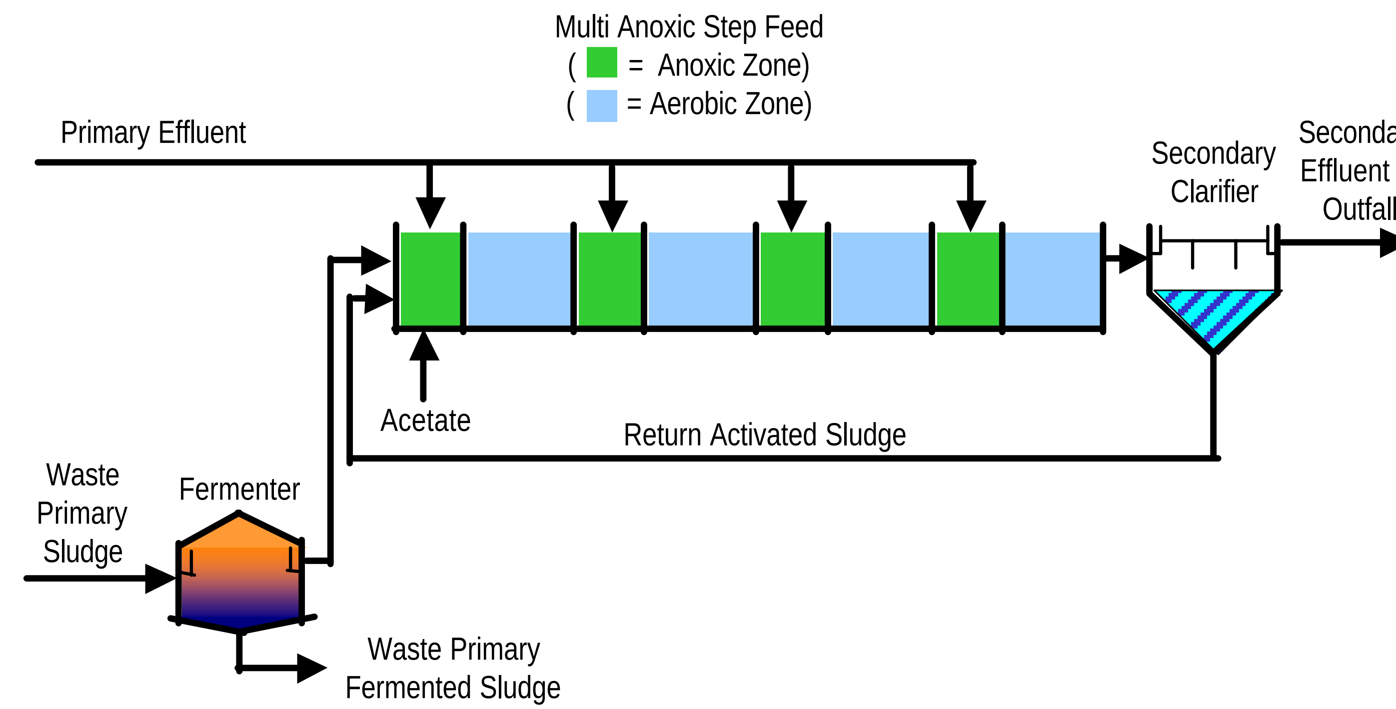
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately 60,000 m^2 + solids handling upgrade
LGWWTP: Approximately 13,000 m^2 + solids handling upgrade

Schematics



Advantages

- Interim LWMP effluent goals met - BOD, TSS
- Odour control requirement limited
- Good quality biosolids
- Ammonia-N removal and aeration saving than CAS

Disadvantages

- Significant plant footprint
- Additional staff and more sophisticated control required
- Land space limitation at LGWWTP
- Different process than AIWWTP and LIWWTP
- Capital costs higher than CEP
- Custom design configuration and control strategy

Comment

- Ammonia-N removal is not required at both plants
- Space limitation at LG WWTP

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The multi anoxic step feed process has been developed and tested in pilot and full-scale exercises in New York City, Cumberland County, Maryland, and Moreno Valley, California for the past ten years. High quality of effluent can be achieved and cost savings are also possible. However, the optimized conditions of this application are highly subject to the wastewater characteristics and operational conditions. The required total bioreactor volume of the multi feed process is considerably smaller than that required for the conventional activated sludge plug-flow process. A typical MLSS concentration in the multi feed process is between 2,500 ~ 5,000 mg/L; the SRT is controlled between 6 ~ 18 days. The step feed ratio among the anoxic zones needs to be custom determined in each application to maximize the benefits.

The DO carryover from the aerobic to anoxic zones is critical to the biological reactions. Dynamic DO control through the multiple stages is the key for a successful operation. Foaming and sludge bulking may be caused, due to the high MLSS concentration in the process, or it may not be a concern due to the multiple-feed setup (optimized F/M ratio). Pilot study is recommended to determine the critical design criteria, including step feed ratio, MLSS concentration, SRT, recycle rate etc. to achieve the desired effluent quality and system stability. The sludge production is expected to be less than the conventional activated sludge process because of the longer SRT.

4.2.5.2 Proven Technology

This process has been developed and applied by many municipal wastewater treatment plants in New York, Maryland and California, to achieve a high rate of total nitrogen removal. However, sophisticated operation and equipment complexity may have compromised its popularity to a certain degree. Custom determined design and operational criteria are necessary for each application.

4.2.5.3 Discharge Requirement/Effluent Quality

The MASF process is capable of achieving a high rate of total nitrogen removal with proper operational control, although nitrogen removal is not a requirement at IIWWTP and LGWWTP. The effluent quality is expected to be 45 mg/L of BOD_5 and 45 mg/L of TSS with 95 % BOD_5 and TSS removal efficiencies. The total N removal efficiency is expected to be 80% resulting in ammonia-N of less than 5 mg/L and total nitrogen of less than 10 mg/L.

4.2.5.4 Reliability

The MASF process is capable of treating influent shock loads with proper step feed arrangement. The process is considered reliable in achieving high standard of effluent quality and total nitrogen removal. Similar to other biological treatments, the MASF process is subject to foaming, filamentous infestations, sludge bulking, temperature variances, and other system upsets. A higher level of operational attention is required to maintain the process stability, as well as the desired removal efficiency.

4.2.5.5 Site Suitability

The footprint required for the MASF process is smaller than that required for an equivalent conventional activated sludge process. The MASF process requires approximately 60,000 m^2 and 13,000 m^2 of footprint expansion at IIWWTP and LGWWTP, respectively, plus solids handling upgrade. It is suitable for IIWWTP to achieve the secondary build-out capacity and performance. However, space may be a constraint at LGWWTP. High demands of operational attention to control the feed ratios, aeration, and internal recycle are anticipated. Due to high level of operational attention and no nitrogen removal requirement, MASF is not recommended for either IIWWTP or LGWWTP.

4.2.6 Pre-anoxic Activated Sludge

4.2.6.1 Process Description

The pre-anoxic activated sludge process is commonly known as the modified Ludzack-Ettinger (MLE) process. The MLE process has been successfully used worldwide to achieve nitrification and denitrification by minimum modification of the conventional complete-mix activated sludge system. A process schematics and technical information are summarized in Figure 4.11. The process is designed to maximize the use of organic substrates for carbonaceous BOD removal and denitrification. The primary effluent can supply the carbon source required for the heterotrophic denitrification in the anoxic zone and the return activated sludge (RAS) and internal recycling will maintain the desired SRT in the system to sustain the nitrifier and denitrifier cultures.

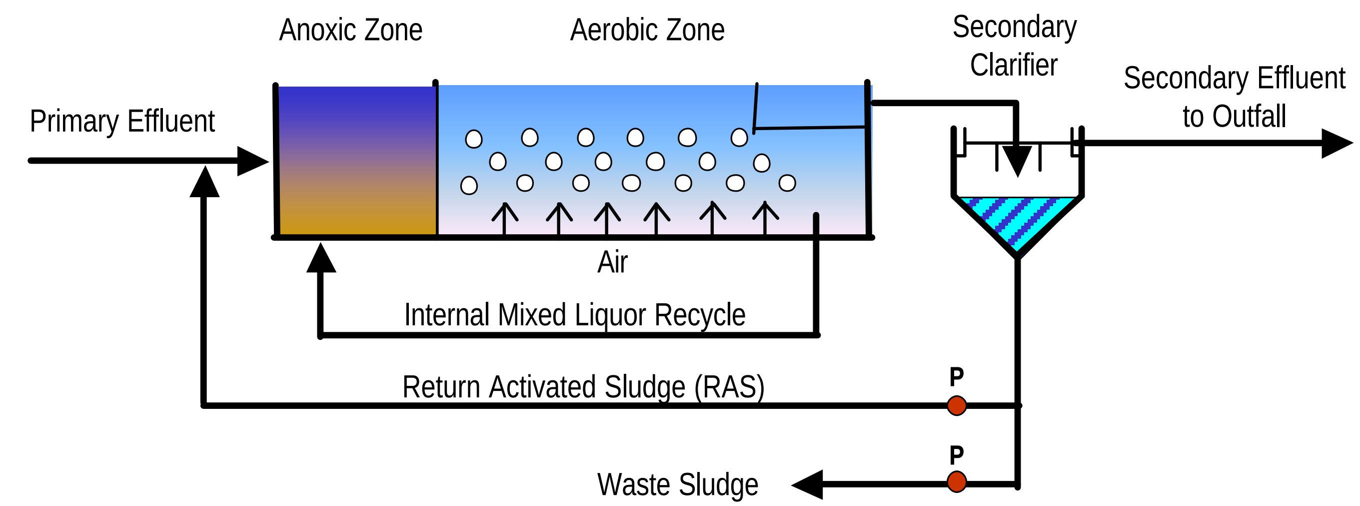
This process is considered advantageous in achieving high quality of effluent (including total nitrogen removal) and saving aeration power (due to denitrification credit). Typically, an anoxic zone (approximately 30% of the aeration basin volume) is added upstream of the AS aeration basin, and internal mixed liquor is recycled from the aerobic zone to the anoxic zone at about 200% - 400% of influent flow for denitrification enhancement. Mixing power needs to be provided in the anoxic zone to keep the MLSS suspended. Proper aeration system needs to be supplied in the aeration zone to maintain the DO level at about 2 mg/L. Solids and MLSS are settled in the following secondary clarifiers to achieve TSS removal.

FIGURE 4.11
PRE-ANOXIC ACTIVATED SLUDGE - MODIFIED LUDZACK ETTINGER (MLE)

Process Description

The process has been successfully used worldwide to achieve nitrification and denitrification by minimum modification of the conventional complete-mix activated sludge system. This process is considered advantageous in achieving high quality of effluent (including total nitrogen removal), and saving aeration power (due to denitrification credit). Typically, an anoxic zone (approximately 30% of the aeration basin volume) is added upstream of the aeration basin, and internal mixed liquor is recycled from the aerobic zone to the anoxic zone at about 200% - 400% of influent flow for denitrification enhancement. Mixing power needs to be provided in the anoxic zone to keep the MLSS suspended, and proper aeration system needs to be supplied in the aeration zone to maintain the DO level at about 2 mg/L. Solids and MLSS are settled in the following secondary clarifiers to achieve TSS removal.

Schematics



Design Criteria

DO in aerobic zone: 2 mg/L
Internal Recycle/total flow: 200 - 400%
SRT: 7 - 15 days
MLSS: 2,000 - 3,000 mg/L
Tank depth: 4.5 - 5.0 m
Anoxic Zone/Aerobic Zone Volume: ~ 0.33
Mixing in anoxic zone: ~10 HP/1000m^3
Secondary settling tank SOR: ~18 m^3/m^2 day
Solids loading rate: ~5.0 kg/m^2 day

Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately 76,000 m^2 + additional solids handling capacity
LGWWTP: Approximately 18,000 m^2 + additional solids handling capacity

Advantages

- Proven technology
- Can handle influent shock load
- Stable sludge condition and reliable effluent quality
- Achieve high effluent quality
- More stable than CAS
- Higher capital cost than CAS
- Aeration saving

Disadvantages

- Still subject to foaming, filamentous infestations, sludge bulking, but better than in CAS and HRAS
- Additional staff required
- Requires higher level of control than CAS
- Different process than AIWWTP and LIWWTP

Comment

- Ammonia-N removal is not required at both plants
- This process is suitable for IIWWTP to achieve the secondary treatment objectives, however it may pose limitation for LGWWTP due to the space constraint

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Additional bioreactor volume (approximately 30% more than the equivalent CAS process), mixing equipment, and recycle pumps are required to facilitate the MLE process. The SRT is maintained at about 7-15 days and the MLSS concentration in the aerobic zone is about 2,000 to 3,000 mg/L, subject to varying conditions of temperature and load. Higher SRT and MLSS concentrations are usually maintained under winter conditions. Alkalinity supplements (e.g. lime) may be necessary to satisfy the denitrification demand (particularly low alkalinity in the Lower Mainland sewage). The addition of a readily biodegradable carbon source, or the operation of a primary sludge fermenter may also be required to provide for denitrification needs. The extra power and chemical demand may be compensated for by the saving in aeration costs. Sludge production is expected to be marginally less than that of the CAS process. However, due to N_2 gas production, sludge settleability in the MLE process is probably poorer than that associated with the CAS process.

4.2.6.2 Proven Technology

The MLE process has proven successful worldwide in many configurations to achieve carbonaceous BOD and biological nutrient removal. High standard of effluent quality and system stability can be maintained with proper process control. The Baltimore City Back River WWTP in Maryland has completed an upgrade to include a 370 ML/d MLE treatment train in 1998 to meet the effluent nitrogen criteria. Examples of the MLE process in Northwest America can be found at Spoken WWTP and Olympia WWTP, Washington.

4.2.6.3 Discharge Requirement/Effluent Quality

The MLE process is capable of delivering high effluent quality with proper operational control. The effluent quality is expected to be 45 mg/L of BOD_5 and 45 mg/L of TSS with 95 % BOD_5 and TSS removal efficiencies. The total N removal efficiency is expected to be 80% resulting in ammonia-N of less than 5 mg/L in the effluent through biological nutrient removal. It should be noted that nitrogen removal is not a requirement at IIWWTP and LGWWTP; therefore, this process is not recommended for either IIWWTP or LGWWTP.

4.2.6.4 Reliability

The MLE process is capable of treating influent shock loads within a certain design range. The process is considered reliable in achieving a high standard of effluent quality and biological nutrient removal. Similar to other biological treatments, the MLE process is subject to foaming, filamentous infestations, sludge bulking, and other system upsets. The MLE system is considered more stable than other conventional activated sludge processes due to diversified microorganism cultures. However, the process requires higher level of control than the conventional activated sludge processes, therefore, it may take longer to reach a steady state condition.

4.2.6.5 Site Suitability

For the design secondary build-out capacity, footprints required for the MLE process are approximately 76,000 m^2 at IIWWTP and 18,000 m^2 at LGWWTP, plus additional sludge handling capacity. The capital investment of the aeration system is significantly higher than that of conventional activated sludge processes because of the extra tankage, mechanical equipment and piping required. This process is suitable for the achievement of secondary treatment objectives at IIWWTP. However, because of space constraints, it may not be a suitable process for LGWWTP.

4.2.7 Sequencing Batch Reactor

4.2.7.1 Process Description

The Sequencing Batch Reactor (SBR) is a fill-and-draw activated sludge treatment system. Aeration and sedimentation / clarification are carried out sequentially in the same tank.

SBR systems usually have five steps that are carried out in sequence as follows:

1) Fill
2) React (Aeration)
3) Settle (Sedimentation/Clarification)
4) Draw (Decant)
5) Idle

A process schematic and summarized technical facts are presented in Figure 4.12.

The two types of SBR systems in use today for domestic wastewater treatment are:

1) Intermittent feed and intermittent discharge (IFID), and
2) Continuous feed and intermittent discharge (CFID).

IFID is the only type suitable for large scale plants. For this reason, CFID will not be discussed.

If a continuous waste stream is to be treated, more than one SBR reactor and/or an equalization tank is required, since a single IFID SBR can only accept influent flow during the fill step. At the end of the fill step in the first reactor, the plant influent must be diverted to a second reactor. Depending on the design, aeration and/or mixing may be provided during the fill step, as well as during the react step. The end of the fill step is usually controlled either by time or by the water level. The discharge flow rate from SBRs during the decant step can be several times the influent flow rate during the fill step, depending on the relative durations of fill and decant; this must be accounted for in the downstream hydraulic design.

Process Description

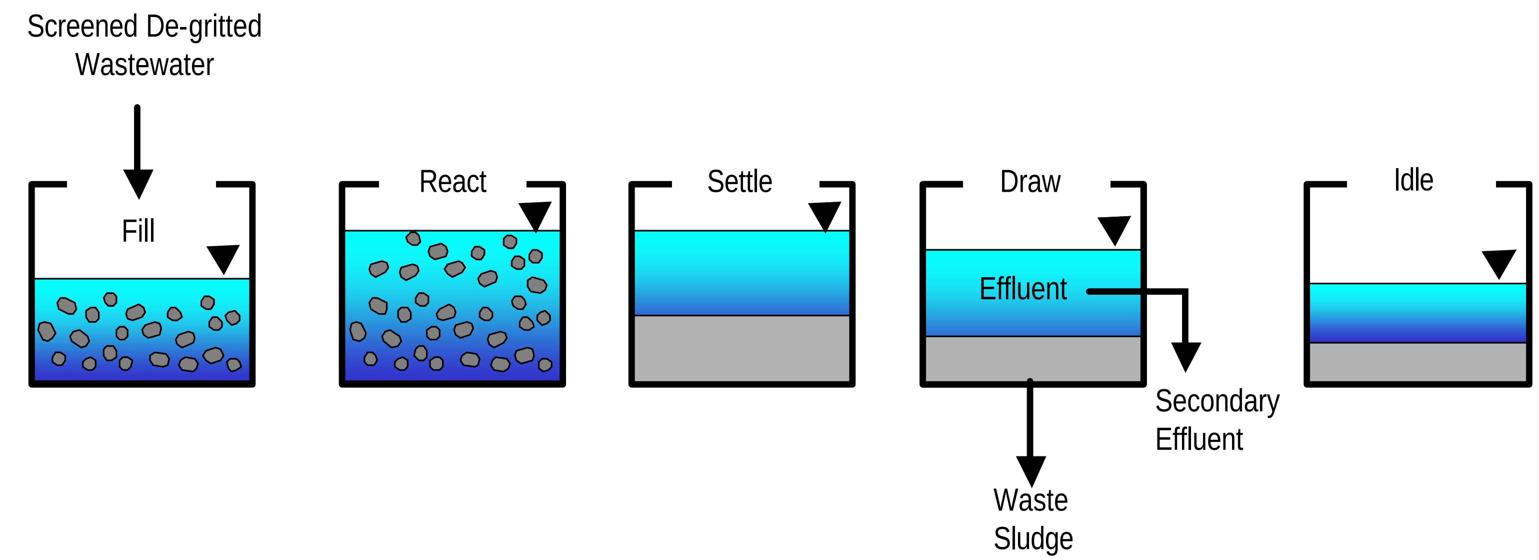
The sequencing batch reactor is a fill-and-draw activated-sludge treatment system. Mixed liquor remains in the reactors during all cycles.

The operation processes are carried out sequentially in the same tank and usually five steps are involved:

1) Fill - wastewater enters bioreactor
2) React - mixing and aeration
3) Settle (sedimentation/clarification) - no mixing or aeration
4) Draw (decant) - removal of treated clarified effluent and settled waste sludge
5) Idle.

Usually a number of tanks are constructed and operated in parallel to address the continuous sewage flow. The reactor can treat either unsettled or settled wastewater.

Schematics



Design Criteria

(Unsettled wastewater)
F/M ratio: 0.04 - 0.10 kg BOD/kg MLVSS.d
SRT: 10 - 30 d
MLSS: 2000 - 5000 mg/L
Volumetric loading: 0.1 - 0.3 kg BOD/m^3.d

Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

May be accommodated on Lions Gate site

Advantages

- Well proven
- No need for separate secondary sedimentation tanks or return sludge pumping
- Provides the ability to control HRT
- Effectively handles varying flows
- Can be operated as a selector process to minimize sludge bulking potential
- Applicable for a variety of plant sizes

Disadvantages

- Complicated process control
- Higher maintenance skills required for instruments, monitoring devices, and automatic valves
- Multiple reactors required for continuous influent stream

Comment

- Multiple reactors required - may not be suitable for LGWWTP because of site constraint
- Only small-scale applications in North America - not yet proven for large flow plant like IIWWTP

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Sludge wasting is an important step in the SBR operation that affects performance. In an SBR operation, sludge wasting usually occurs during the settle or idle phases. There is no need for a return activated-sludge (RAS) system. This is because both aeration and settling occur in the same chamber.

The key to the SBR process is the control system, which consists of a combination of level sensors, timers, and microprocessors. Programmable logic controllers can be configured to provide a precise and versatile means of control.

4.2.7.2 Proven Technology

SBRs came into existence in early 1960s with the development of the new technology and equipment. All wastewater commonly treated by conventional activated-sludge plants can be treated by SBRs. The system has been successful at a variety of USA and worldwide installations with flow rate as high as 190 ML/d in one UK installation. However, most SBR plants in the North America are designed to handle a flow of less than 40 ML/d, lower than that of either IIWWTP or LGWWTP.

4.2.7.3 Discharge Requirement/Effluent Quality

With appropriate design and operation, SBR plants have been reported to produce high quality BOD and TSS effluents. Typical ranges of BOD_5 are from 5 to 15 mg/L. TSS can range from 10 to 30 mg/L in well-operated systems. The removal efficiencies of BOD_5 and TSS are both greater than 90%.

4.2.7.4 Reliability

Because multiple batch tanks are used, it is unlikely that the entire plant would be effected by a shock load. Plant recovery is assisted by the availability of an inventory of sludge in multiple tanks.

4.2.7.5 Site Suitability

Existing primary treatment facilities would continue to be used. Multiple reactors are required to address the large flows at IIWWTP and LGWWTP. Availability of space is a concern at Lions Gate.

4.2.8 Membrane Activated Sludge

4.2.8.1 Process Description

Membrane Activated Sludge (MAS) consists of a biological reactor with suspended biomass and liquid-solid separation by microfiltration membranes with nominal pore sizes ranging 0.1 to 0.4µm. Biological treatment of the wastewater is carried out by the activated sludge system. The treated water is separated from the active sludge by a process of membrane filtration rather than in a secondary clarifier as in conventional systems. Only the treated effluent is drawn through the membrane by pumping. The sludge is recovered and dewatered. A process schematic and summarized technical facts are presented in Figure 4.13.

There are two types of membrane modules currently in use:

1) Submerged membrane module (usually Hollow fibre); and
2) Cross-flow membrane module.

MAS systems may be used with aerobic or anaerobic suspended growth bioreactors to separate treated wastewater from the activated sludge. The sludge concentration (10-40 kg MLSS/m^3) and reactor capacity (10-50 kg BOD/m^3/d) are very high in membrane biological treatment (up to 40 times higher than in conventional treatment).

By replacing the traditional secondary clarifier gravity solids separation with membranes avoids issues of filamentous sludge bulking and other floc settling and clarifier problems. Additionally, the MLSS concentration is no longer controlled by the secondary clarifiers. However, membrane fouling might occur. This is because the biomass coats the outer layer of the effluent during withdrawal. Finer particles may penetrate the inner pores of the membrane, causing an increase in pressure loss. Continuous cleaning is required to reduce the fouling problem.

4.2.8.2 Proven Technology

MAS is a relatively new technology in the area of municipal wastewater treatment. An example of a North American installation is the City of Traverse City, Michigan, USA.

4.2.8.3 Discharge Requirement/Effluent Quality

The MAS process can produce an effluent with a BOD_5 of less than 5 mg/L and turbidity of less than 1 NTU. In fact, low effluent BOD and turbidity concentrations are possible for MAS systems with MLSS concentrations in the range of 6,000 to 16,000 mg/L. The system should comfortably meet the 45/45 BOD/TSS secondary effluent standard and also comfortably meet MBAS removal to <2 mg/L.

4.2.8.4 Reliability

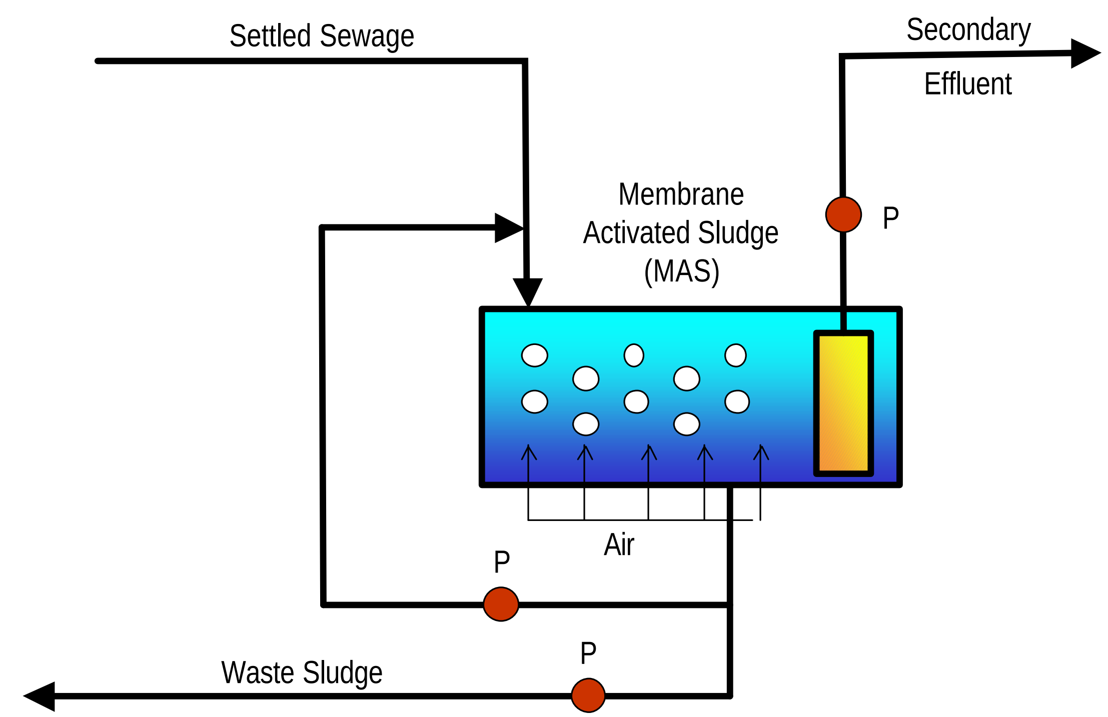
Membrane fouling may occur from time to time with MAS treatment and therefore proper control is required. The fouling problem makes it uneconomical to employ MAS treatment in large wastewater treatment systems and therefore it is not suitable for use at IIWWTP because of its high wet weather flow.

FIGURE 4.13
MEMBRANE ACTIVATED SLUDGE (MAS)

Process Description

This process is similar to the conventional activated sludge (CAS) process except that a membrane system (Ultrafiltration or Microfiltration) replaces the final settling tank. Primary effluent from the existing primary settling tanks at Iona and Lions Gate will be discharged to the activated sludge system. The activated sludge organisms utilize organics in wastewater as a food source and convert them to AS cells, carbon dioxide and water. Compressed air is applied to the bioreactor to maintain the micro-organisms in an aerobic condition. The effluent is extracted through membrane filters. The membrane can be submerged within the bioreactor (as shown in the schematics) or placed in-series downstream of the bioreactor.

Schematics



Design Criteria

F/M: <0.2 kg BOD_5 / kg MLSS d
Sludge age: > 15 days
MLSS: 10,000 mg/L
Volumetric Loading: 1.2 – 3.2 kg COD/m^3d
HRT: > 2 hours
Air requirements: 1.2 kg O_2 kg BOD_5 removed

Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Should be accommodated on the Iona and Lions Gate sites

Advantages

- Much Smaller Footprint than CAS
- Remove Pathogenic Organisms, Provide Disinfection
- Good Quality Biosolids
- LWMP Effluent Goals Met - BOD, TSS
- Toxicity Reduced to LC_{50} of 100% Effluent

Disadvantages

- Membrane Fouling Problem
- Limited Track Record
- High Energy Consumption
- Capital Costs High in Comparison with CAS
- Higher Operating Cost than CAS (Membrane Replacement)
- Membrane Life Questionable
- Different Process than Annacis / Lulu

Comment

- Good option for Lions Gate for the small footprint required
- Not an economic option for Iona

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4.2.8.5 Site Suitability

The combination of activated sludge system and settling system renders MAS a much smaller plant footprint than conventional treatment system. This favours the installation at LGWWTP where the lack of real estate is a concern.

4.2.9 Deep Shaft Technology (Vertreat®)

4.2.9.1 Process Description

The Vertreat® System is a proprietary, high-rate activated sludge sewage treatment process. A process schematic and summarized technical facts are presented in Figure 4.14.

In this process, the aeration basin typical of most activated sludge processes is replaced by a vertical shaft 120 m to 150 m deep. The shaft is lined with a steel shell and fitted with a concentric central pipe to form an annular reactor. The central pipe is called the downcomer, and the annular stack is called the upcomer. Compressed air is introduced into both the downcomer and the upcomer, to provide oxygen for biological activity, and to provide the driving force for fluid circulation. The process mixed liquor and the influent wastewater are forced down the central pipe (downcomer); the aerated mixture descends to the bottom of the shaft, and returns to the surface via the annulus (upcomer).

Under the high pressures prevailing at the bottom of the shaft, the solubility of oxygen in water is increased, allowing a higher concentration of dissolved oxygen in the process mixed liquor, compared to conventional activated sludge processes. Effluent is withdrawn from the reactor bottom at high velocity, preventing grit deposition and routed to a flotation clarifier. Under atmospheric pressure in the flotation clarifier, the solubility of oxygen in water is reduced, and the solution becomes supersaturated with oxygen. Dissolved oxygen forced out of solution forms bubbles, which attach to the activated sludge flocs (clumps), and carry them to the surface. Most of the floating biological solids are returned to the shaft, and some are wasted to the biosolids treatment system.

4.2.9.2 Proven Technology

The process concept of Vertreat®, deep shaft, is a proven technology employed in more than 80 plants worldwide. While most of the Vertreat® applications in municipal wastewater treatment are in Japan and UK, an example of Vertreat® installation in North America can be found in the City of Homer, Alaska, USA. It is, however, of a much smaller scale than either IIWWTP or LGWWTP.

4.2.9.3 Discharge Requirement/Effluent Quality

The effluent BOD_5 and TSS after the Vertreat® process are expected to be both less than 10 mg/L, which comply with the permit. The process should comfortably meet the 45/45 BOD/TSS secondary effluent standard.

FIGURE 4.14
VERTREAT-U-TUBE TECHNOLOGY

Process Description

The Vertreat U-Tube technology is a variation of the high-rate activated-sludge process. A vertical shaft, approximately 120 to 150m deep, replaces the aeration basin. The shaft is lined with a steel shell and fitted with a concentric pipe to form an annular reactor. Mixed liquor and air are forced down the centre of the shaft and allowed to rise upward through the annulus. The mixed liquor returning to the surface is supersaturated with oxygen. Oxygen bubbles rise in a flotation tank, carrying the process solids to the surface, where they are removed by a skimmer.

Design Criteria

Volumetric Loading: 3.0 - 6.0 kg BOD/day/m^3 reactor vol.
Organic loading: 0.75 - 1 kg BOD/day / kg MLVSS
Sludge age: 2.5 - 5 days
RAS rate: 0.2 - 0.5
HRT: 0.75 - 2.5 hours
Air requirement: 4.5 - 6.5 Nm^3/kg BOD (Delivered at up to 10 atm, typical energy use is 0.5 - 0.9 kW hr / kg BOD removed)

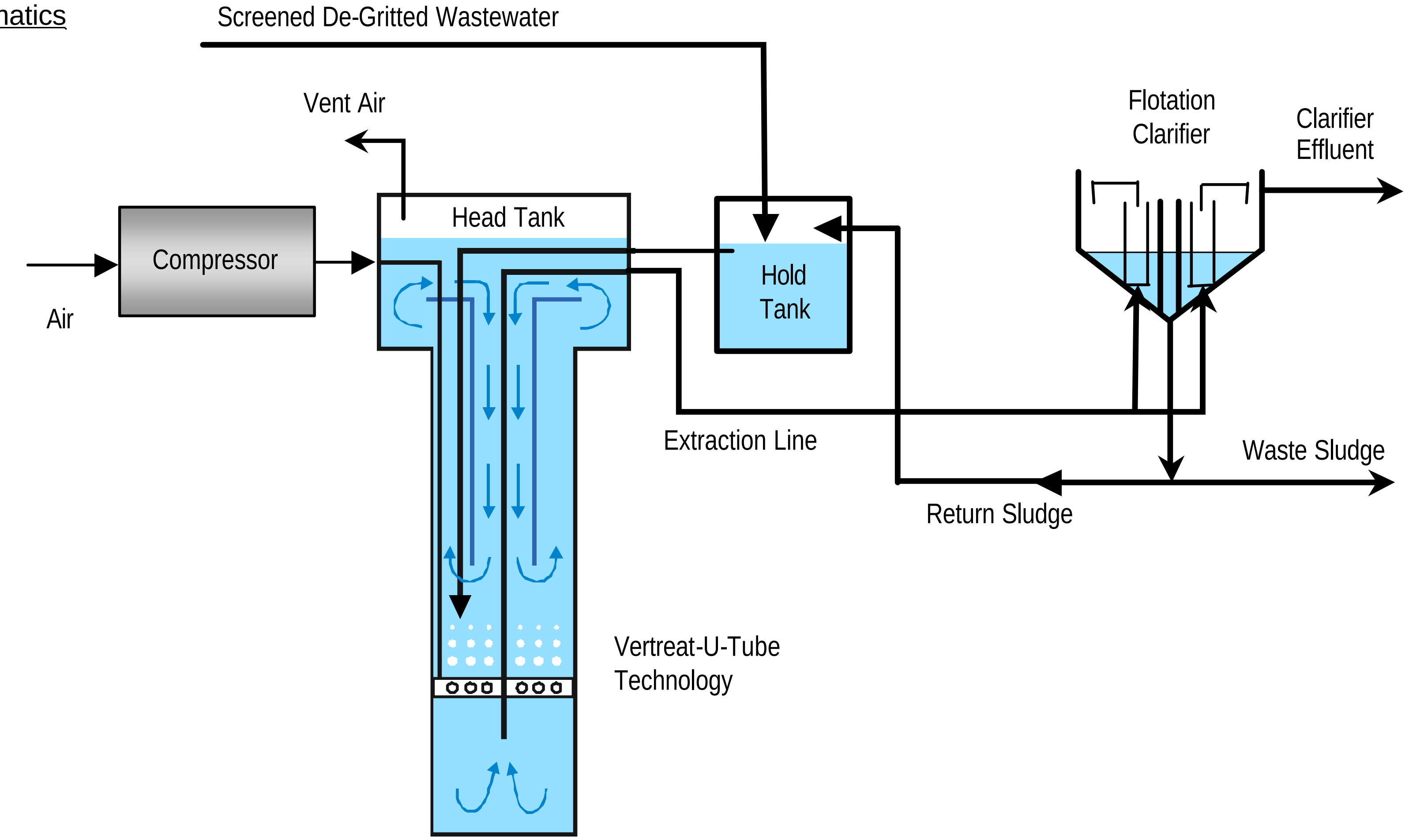
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Should be accommodated on the Iona and Lions Gate sites

Schematics



Advantages

- Small plant footprint: 10 to 20% of space required by conventional processes
- Few odour or odour control problems
- Not affected by atmospheric temperature
- Can handle strong industrial wastes

Disadvantages

- Relatively unproven
- Construction of shaft may have high cost, depending on site-specific conditions

Comment

- Good option for Lions Gate for its small plant foot print
- Only have small-scale application, not yet proven for large system like IIWWTP

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4.2.9.4 Reliability

The process is unaffected by climatic changes. In addition, because the process includes complete mixing, it has the ability to handle shock loads very well.

4.2.9.5 Site Suitability

The Vertreat® system requires only 10 to 20% of space required by conventional activated sludge process and does not require primary treatment. The small plant footprint would be an advantage at LGWWTP where site constraint is a concern.

4.2.10 Upflow Sludge Blanket Filtration Clarifier (USBF)

4.2.10.1 Process Description

The USBF process is a proprietary modification of conventional activated sludge process that incorporates an anoxic selector zone and an upflow sludge blanket clarifier. The USBF process can be designed for (1) carbonaceous BOD removal, (2) BOD removal and nitrification, (3) BOD removal, nitrification, and denitrification, and (4) BOD removal, nitrification/denitrification and phosphorus removal. A process schematic and summarized technical facts are presented in Figure 4.15.

In operation, wastewater enters the anoxic compartment of the bioreactor where it mixes with activated sludge recycled from the bottom of the clarifier. Agitated and moving in a plug flow manner, the mixed liquor underflows into the bioreactor's aerobic compartment.

After aeration, a stream of the mixed liquor enters the bottom of the clarifier where sludge flocs and water are separated by upflow sludge blanket filtration. After separation, clear water overflows into a collection trough and is discharged from the system. To complete the internal circulation loop, activated sludge collecting at the bottom of the clarifier is recycled back to the bioreactor's anoxic compartment.

The USBF process includes a unique patented upflow sludge blanket clarifier. The upflow blanket clarifier has a trapezoidal shape. Mixed liquor enters the bottom of the clarifier through a specially designed baffle where hydraulically induced flocculation occurs. The shape of the clarifier provides a steadily increasing surface area from the bottom to the top. As a result, there is a gradually decreasing vertical velocity gradient within the clarifier. The "top surface area" clarifier overflow rate is 6 to 10 $m^3/d/m^2$ at the average daily design flow. The clarifier is typically designed for a daily peak flow rate of 3 times the average flow ratio. This is equivalent to a peak "top surface" clarifier overflow rate of 18 to 31 $m^3/d/m^2$. This is a very conservative overflow rate.

FIGURE 4.15
UPFLOW SLUDGE BLANKET FILTRATION CLARIFIER (USBF)

Process Description

The USBF process combines the anoxic zone, aerobic zone and clarifier into one tank. Primary effluent from the existing primary settling tanks will be discharged to the anoxic compartment of the bioreactor where influent BOD serves as a carbon source for reducing nitrate (denitrification). The MLSS will then underflow into the aerobic compartment where BOD removal and nitrification take place. After aeration, the MLSS will enter the bottom of the clarifier. Treated effluent will flow to the top of the clarifier and be discharged while the activated sludge is settled to the bottom of the clarifier and recycled back to the anoxic compartment.

Design Criteria

F/M: 0.01 to > 1 kg BOD_5 / kg MLSS d
Sludge age: 5 - 70 days
MLSS: 4,000 - 6,000 mg/L
SVI: 80 - 120
RAS rate: 25 - 100% Q
HRT: 1 - 2 hours for C removal, 2-8 hours for nitrification and denitrification (Anoxic zone), 6 - 30 hours (Aerobic zone)
DO requirements: < 0.2 mg/L (Anoxic zone), 2 - 4 mg/L (Aeration zone)
Clarifier surface overflow rate: 18 - 31 m^3/m^2 day (peak Q)

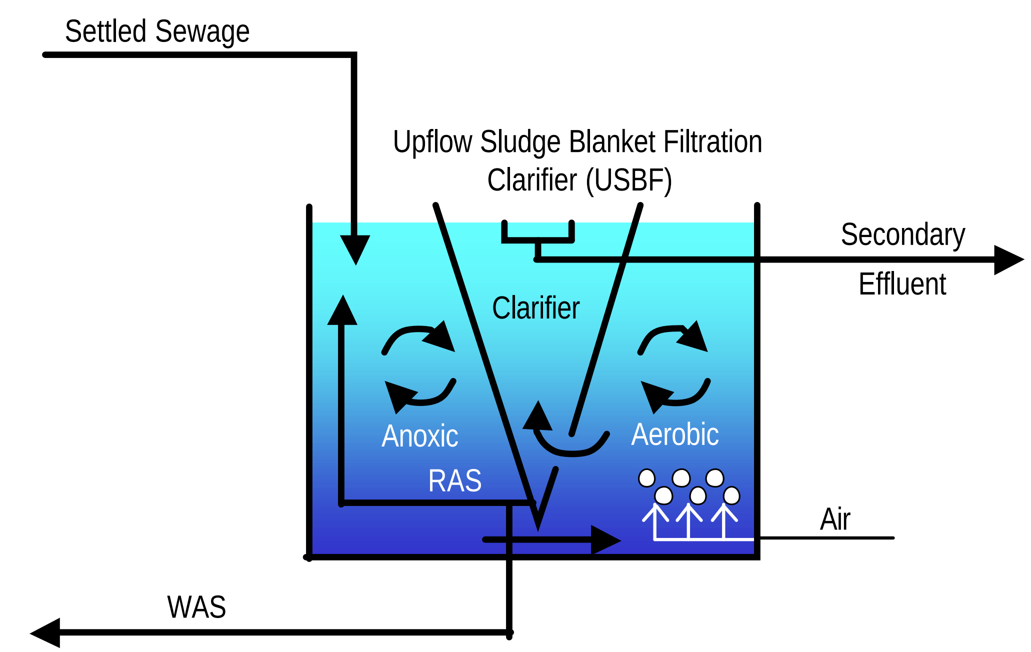
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 95%
TSS mg/L	> 95%

Plant Footprint

Unlikely to be accommodated on Lions Gate site.

Schematics



Advantages

- Smaller Footprint than CAS
- LWMP Effluent Goals Met - BOD, TSS
- Toxicity Reduced to LC_{50} of 100% Effluent
- Little Odour
- Good Quality Biosolids

Disadvantages

- Proprietary Process
- Capital Costs High in Comparison with CAS
- Different Process than Annacis / Lulu
- Only Small Scale Systems in North America

Comment

- Not commonly used by plants of size of IIWWTP or LGWWTP
- Footprint makes it problematic to be used at LGWWTP

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The clarifier also includes a unique baffle arrangement to allow sludge withdrawal at the bottom of the clarifier. The sludge withdrawal system design also incorporates internal recycle between the aerobic and anoxic zones.

4.2.10.2 Proven Technology

Process concepts incorporated into the patented USBF process were developed both in Europe and the U.S. in the 1970's. Various components of this process including "anoxic selector zones", and "upflow blanket clarifiers" have been used worldwide for the last 25 years. However, the history of the patented USBF system itself is unknown. The USBF system is in operation in thousands plants worldwide but in North America the systems are mostly small-scale. The process is unsuitable for large-scale systems such as IIWWTP and LGWWTP.

4.2.10.3 Discharge Requirement/Effluent Quality

The removal efficiencies of BOD_5 and TSS in raw influent are greater than 95%. The process should comfortably meet the 45/45 BOD/TSS secondary effluent standard.

4.2.10.4 Reliability

The USBF process responds well to peak to average hydraulic loading. An increase in hydraulic loading will, due to the lower Sludge Volume Index (SVI), result in a faster settling mixed liquor**.** The sloping sidewall clarifier will then allow the sludge blanket to rise. As a consequence, the surface settling area will increase. The inter partial flocculation in the upflow clarifier also helps the process to perform well under fluctuating flow conditions.

4.2.10.5 Site Suitability

Although the USBF single tank systems have been installed with up to 4.0 mgpd (15,000 m^3/d) capacity, this is a much smaller scale than either LGWWTP or IIWWTP. Installing multiple USBFs in LGWWTP would not be feasible because of the lack of real estate.

4.3 ANAEROBIC PROCESSES

4.3.1 Process Description

Anaerobic processes achieve the breakdown of inorganic and organic material chemically and biologically in the absence of oxygen. They are the oldest process used for sludge stabilization and remain the most commonly applied unit operation. The organic materials are converted to carbon dioxide, hydrogen, water and methane at different stages and under different conditions. The performance of an anaerobic treatment process is highly dependent on the operational parameters, including pH, temperature, organic load, chemical buffer capacity, and the presence of toxic substances. The most common temperature ranges of anaerobic processes are about 30 ~ 38 ºC (mesophilic anaerobic, i.e. IIWWTP) and 50 ~ 58 ºC (thermophilic anaerobic, i.e. LGWWTP and AIWWTP).

Anaerobic processes are usually utilized in the treatment of wastes with high organic and solids loads. Recently, it has been successfully employed to treat diluted waste, mainly industrial wastewater with recalcitrant (less degradable) constituents. Many anaerobic processes using suspended-growth, attached-growth, or their combination have been commercially developed and operated on a full-scale. Based on their organic load (kg COD/m^3/d), anaerobic treatments are also categorized as high rate, medium rate or low rate processes.

4.3.1.1 (Continuous stirred tank reactor (CSTR) Low Rate Bioreactor

This process consists of a completely mixed anaerobic tank followed by an external clarifier for solids separation. The process schematic and technical facts are summarized in Figure 4.16. A portion of the anaerobic biomass is returned to the completely mixed tank. The CSTR usually consists of a tall circular tank and requires mixers and recirculation pumps to keep the tank contents mixed. External clarifiers are needed as the effluent from the tank has a high solids content. The CSTR process is classified as low to medium rate with a COD loading in the range of 2 to 5 kg/m^3/day. The hydraulic retention time varies from 1 to 3 days. The external clarifier must be covered to conserve heat and prevent odours.

As in all anaerobic treatment plants, the influent must be conditioned before entering the bioreactor in order to raise the temperature, to add nutrients (phosphorus and nitrogen) and to adjust the alkalinity. This can be done in line or in a separate conditioning tank. Flow equalization is also needed. Wastewater high in suspended solids and FOG are appropriate for treatment in a CSTR.

There is also a batch variation of the CSTR which includes filling, reacting, settling and decanting in a single reactor. A separate clarifier is not required. With a batch CSTR, a minimum of two reactors is required, unless a very large equalization basin is provided to hold the wastewater during the reacting, settling and decants phases.

4.3.1.2 Upflow Anaerobic Sludge Blanket (UASB)

UASB is the most common type of high-rate anaerobic process currently in use. Process schematics and technical information are summarized in Figure 4.17. Wastewater is introduced through an influent distribution system at bottom of the reactor and flows upwards through a granular sludge blanket. The sludge blanket is formed from the biodegradation of wastewater, and is composed of dense spherical granular sludge particles. Above the sludge blanket is a clear zone where lighter biomass particles are suspended. At the top of the tank gas/liquid/solid separators or clarifiers are suspended from the roof. Alternatively, the UASB may be followed by an external clarifier.

FIGURE 4.16
CONTINUOUS STIRRED TANK REACTOR (CSTR)

Process Description

The continuous stirred tank reactor (CSTR) in the anaerobic application is covered and has a continuous inflow and outflow of wastewater. Influent that is fed into the reactor is mixed completely before leaving the tank. The concentration within the tank volume is uniform and equals to that of the effluent. The hydraulic retention time and solids retention times are equal for this kind of low rate process. The CSTR bioreactor without sludge recycle is more suitable for wastewater with high concentrations of solids or extremely high dissolved organic concentrations. Various methods of mixing may be used to utilize the full reactor volume.

Design Criteria

Volumetric organic loading: 2 - 5 kg COD/m^3.d
HRT: 1 - 3 days

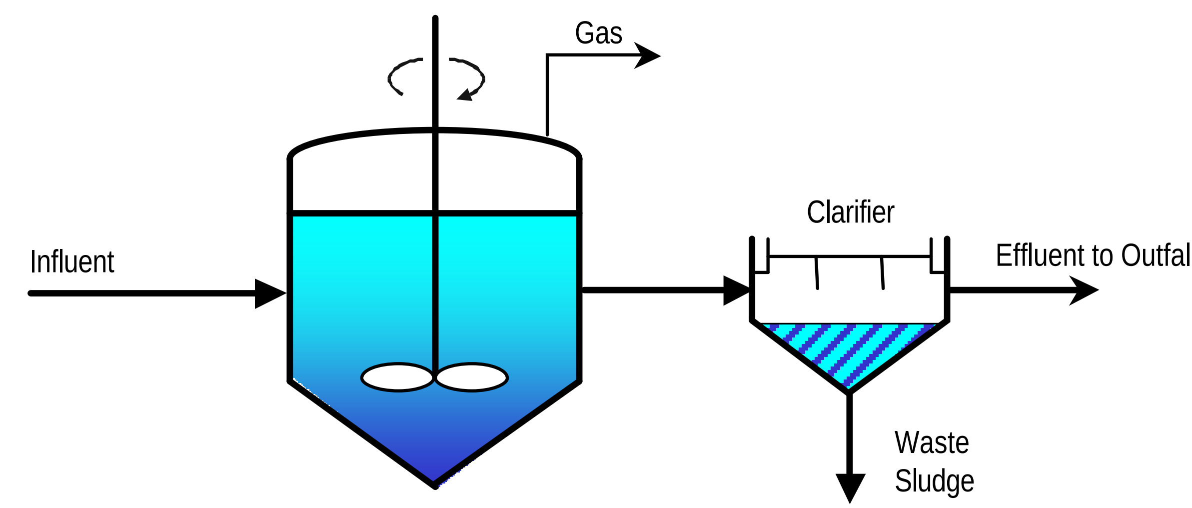
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately: 350,000 m^2 plus sludge handling
LGWWTP: Approximately: 88,000m^2 plus sludge handling

Schematics



Advantages

- Low energy required
- Small reactor volume required thus less space needed
- Methane gas production - source of energy
- Rapid response to substrate addition after long periods without feeding
- Ability to handle shock loads
- Low biomass yield

Disadvantages

- Corrosion of ferrous-metal piping and supports.
- Require solids separation after bioreactor
- Equipment wear by grit.
- Equipment plugging and operational interference by rags.
- May require alkalinity addition
- Biological nitrogen and phosphorus removal is not possible
- Potential for production of odours and corrosive gases

Comment

- Large land space required, not feasible for LGWWTP
  Usually apply to higher strength waste or pre-treatment

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FIGURE 4.17
UPFLOW ANAEROBIC SLUDGE BLANKET (UASB) BIOREACTOR

Process Description

Under an anaerobic condition, the waste is introduced to the bottom of the reactor in a UASB process. The wastewater flows upward through a sludge blanket composed of biologically formed granules or particles. Treatment is characterized by its internal settling compartment. An internal gas-liquid-solid (G-L-S) separator is built in the top of the reactor. The bacteria aggregate in the reactor to form biogranules. The high settleability of biogranules enables the bacteria to return to the reactor beneath the G-L-S separator, despite the high liquid turbulence. As a result, high bioactivity and high biomass concentrations can be maintained in the reactor. Modifications include an external settling tank or the addition of packing material to the top of the sector to prevent major loss of system biomass.

Design Criteria

HRT: 4 - 8 hr for domestic wastewater
Organic loading: 10 - 15 kg COD/m^3/d
Upflow velocity: 0.8 - 1.0 m/hr for domestic wastewater
Depth: 6 - 8 m

Expected Performance

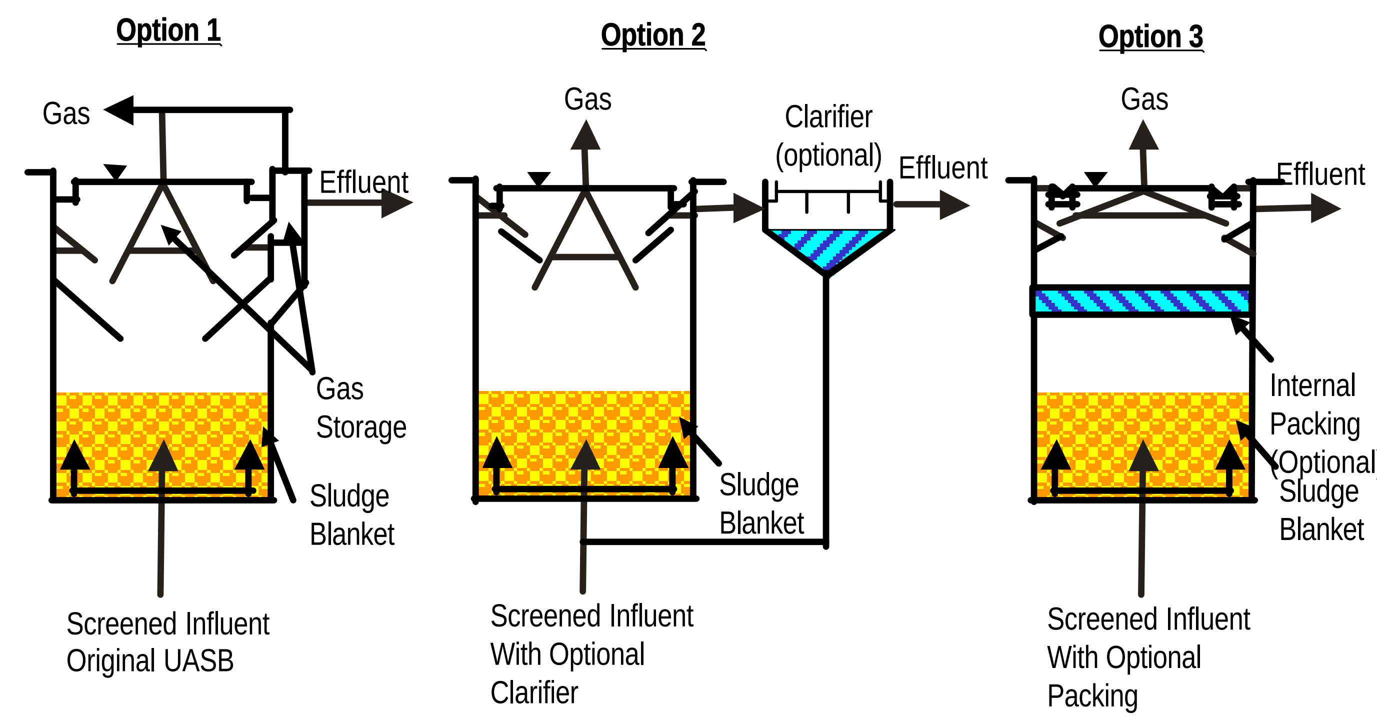
COD removal = 90-95%

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 45

Plant Footprint

IIWWTP: Approximately: 53,000 m^2 plus sludge handling
LGWWTP: Approximately: 15,000 m^2 plus sludge handling

Schematics



Advantages

- Excellent settling characteristics of granular biomass.
- Can handle high organic loading
- Relatively low HRT
- Well proven for industrial wastes

Disadvantages

- Odour control is required.
- Development of granular solids is affected by wastewater character
- Biomass is susceptible to washout
- Typically used for high strength wastewater or pre-treatment

Comment

- Typically used for high strength industrial wastes only, not suitable for IIWWTP and LGWWTP

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This is a high rate process with COD loading rate in the range of 10 to 15 kg/m^3/day and a retention time of 1 to 2 days. Like most high rate processes, UASB are generally used for wastewater with high COD, and low TSS and FOG. Typical applications include wastewater from breweries, the beverage industry, sugar mills, starch factories and paper mills. An equalization basin similar in size to the existing equalization basin must precede the bioreactor.

4.3.1.3 Packed Bed Filter

The fixed film process differs from the other high rate processes. Instead of being in suspension in the liquid the anaerobic microorganisms are attached to a fixed surface with a packed bed. The process schematic and technical facts are summarized in Figure 4.18. The fixed film process has some similarity to the aerobic trickling filter process except that the flow is upward. For this reason the fixed film process is often referred to as an upflow packed bed or upflow filter. Fixed film processes require an influent with maximum TSS concentration of approximately 200 mg/L as TSS in excess of this will foul the filter media. Typical media for fixed film consist of plastic spheres. Typical uses for the fixed film process include the treatment of chemical industry liquid waste with very high COD and very low TSS.

4.3.1.4 Fluidized Bed

The fluidized bed is a modification of the basic UASB process. The process schematic and technical facts are summarized in Figure 4.19. In a fluidized bed, heated and conditioned wastewater is introduced at the bottom of a tall cylindrical vessel. The main distinction is that fine sand is mixed with the influent wastewater and acts as a “seed” on which the anaerobic bacteria grow. The fine particles of sand covered with bacteria are maintained in suspension (fluidized) by at the wastewater flowing in from the bottom of the small diameter vessel.

4.3.1.5 Bulk Volume Fermenter (BVF)

This patented process is marketed by ADI under the trade name of Bulk Volume Fermenter (BVF). The process schematics and technical facts are summarized in Figure 4.20. Wastewater is introduced in the bottom of a very large tank with a geomembrane floating cover to collect the biogas. Wastewater enters the reactor through a pipe network beneath the sludge bed. As the wastewater passes upward through the sludge blanket, anaerobic microorganisms convert the waste to biogas and sludge. A floating settler is located in the tank to separate the clear liquid from the solids. The biogas rises through the liquid and emerges just beneath the geomembrane floating cover and flows to the tank perimeter where it is collected. A small negative pressure is applied, by means of external blowers to prevent escape of biogas into the atmosphere. A sludge recycle system returns sludge to the influent. The same system is used to waste sludge.

Process Description

The packed-bed reactor consists of a container that is packed with a media to which the microorganisms can become attached (rock, slag, ceramic or plastic). The wastewater flows up or down through the interstitial spaces between the packing and biogrowth. Effluent recycle is generally not used for packed bed reactors except for high-strength wastewaters. A variety of designs employing synthetic plastic packing are currently used.

The fixed film processes require an influent with maximum TSS concentration of approximately 200 mg/L. The presence of TSS in excess of 200 mg/L will foul the filter media. Typical media for fixed film consist of plastic spheres. Typical uses for the fixed film process include very high COD liquid waste from the chemical industry where the TSS content is very low.

Design Criteria

Height: 3 - 13 m
Packing material placement may be in the entire depth or only upper 50 - 70 %
Specific surface area of packing: ~100 m^2/m^3
Low upflow velocities are used to prevent washing out the biomass
For cross-flow packing type:
Temp: 30 - 35ºC
COD loading: 1.5 - 6 kg/ m^3.d
HRT: 10 -12 hr

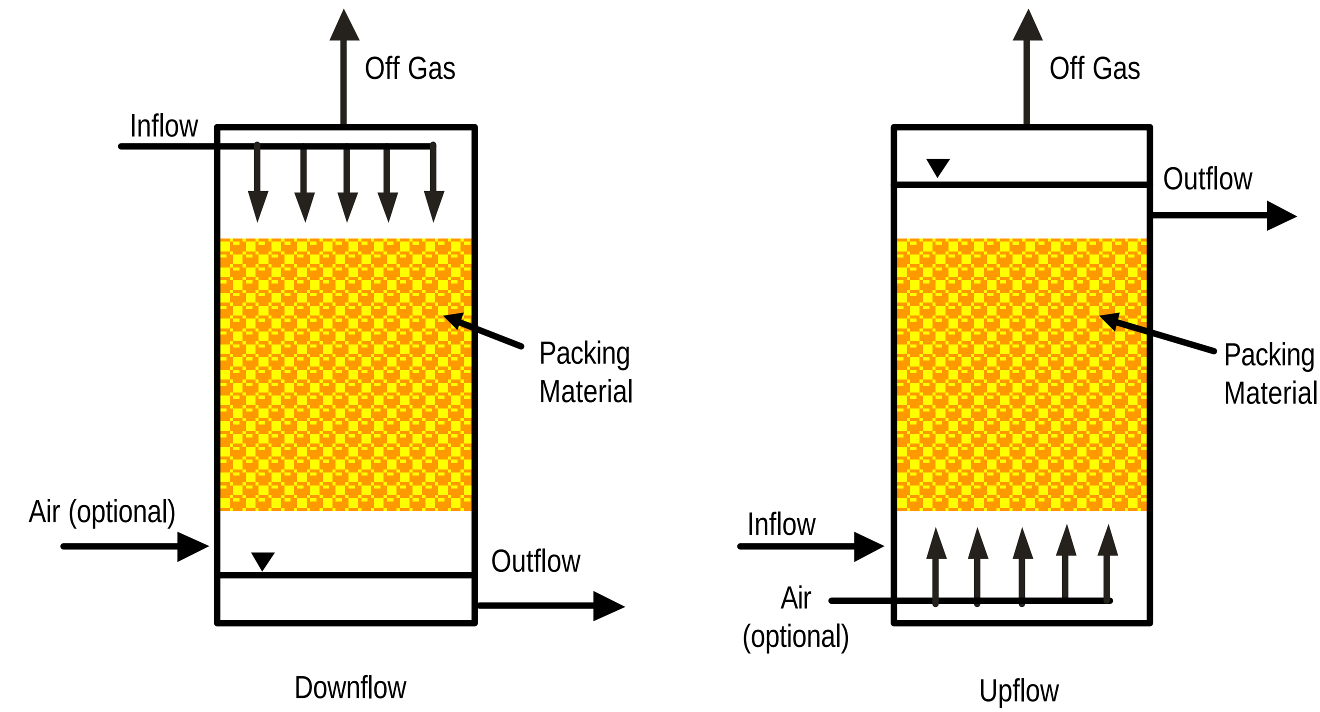
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 50

Plant Footprint

IIWWTP: Approximately: 55,000 m^2 plus sludge handling
LGWWTP: Approximately: 14,000m^2 plus sludge handling

Schematics



Advantages

- High COD loadings
- Relatively small reactor volumes
- Operational simplicity

Disadvantages

- Solids and biomass may accumulate in the packing and cause plugging and flow short circuiting.
- Expensive packing material
- Operational problems and maintenance associated with solids accumulation and possible packing plugging

Comment

- This process is best suited for low suspended solids and high organic concentrations

FIGURE 4.19
FLUIDIZED BED REACTOR

Process Description

The fluidized bed system uses fixed biomass growing on a fluidized carrier material (eg. sand, basalt, pumice, etc.) Wastewater is introduced from the bottom of the reactor through an appropriate underdrain system or inlet chamber. The bed of carrier material is expanded by the upward movement of fluid through the bed. The porosity of the bed can be controlled by varying the upward flow rate of the fluid. The process may be anaerobic, or aeration may be used depending on the process objectives. Packing is removed at the top of the reactor and passed through a high shear pump to separate biomass from the packing. The cleaned packing material is returned to the reactor. Effluent recycle is used to provide sufficient upflow velocity.

Design Criteria

High upflow liquid velocities of about 20 m/h to provide about 100% bed expansion
Packing size: ~0.3 mm sand
Reactor depth: 4 - 6 m
COD loading: 3 - 42 kg / m^3.d
HRT: 3 - 15 hr

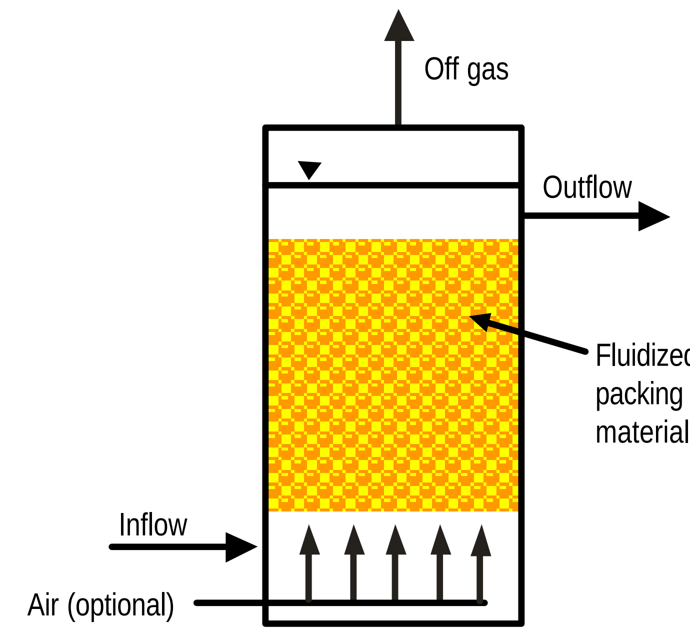
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 50

Plant Footprint

IIWWTP: Approximately: 110,000 m^2 plus sludge handling
LGWWTP: Approximately: 27,000m^2 plus sludge handling

Schematics



Advantages

- Better removal efficiency than conventional complete-mixed reactor due to increase in specific surface area and biomass concentration
- Good mixing and hence ability to handle shock loads
- Good uniformity of temperature
- Operated without the need of effluent filtration or clarification
- Higher treatment performance than packed-bed reactors for higher loadings
- Minimal space requirement

Disadvantages

- Excessive growth on the top part of the reactor and no growth on the carrier in the low part of the reactor
- Lower solids capture than packed bed reactor because of high turbulence and thin biofilms developed
- Extra care for startup in this process compared with other high-rate anaerobic reactors
- Care must be taken in the inlet and outlet designs to ensure good flow distribution
- Pumping power required to operate the fluidized bed
- Cost of reactor packing
- Need to control the packing level and wasting with biogrowth

Comment

- This process is best suited for wastewaters with mainly soluble COD

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BULK VOLUME FERMENTER (ADI-BVFÒ)

FIGURE 4.20
BULK VOLUME FERMENTER (ADI-BVFÒ)

Process Description

The ADI-BVF® is a patented low-rate anaerobic lagoon process. It is typically of earthen and concrete construction with about 3 to 4 m vertical concrete side walls. The reactor has a floating geomembrane cover with a layer of closed-cell polyethylene insulation attached to its underside. The cover allows for collection of biogas, temperature control and positive odour control. Wastewater enters the reactor via a header-lateral pipe network beneath the sludge bed. Recycled sludge mixes with the feed according to the pumping schedule. Interior baffles are provided to promote retention of sludge within the influent zone. Wastewater flows through a series of gas-liquid-solid separators which act as internal clarifiers to inhibit the movement of solids to the reactor surface. The reactor also contains a low-speed mixer which operates on an intermittent basis for short periods of time.

Design Criteria

Maximum liquid depth: 7 - 9 m
COD loadings: max 1 kg / m^3.d
Temperature: 15 - 25ºC
HRT: 7 hr

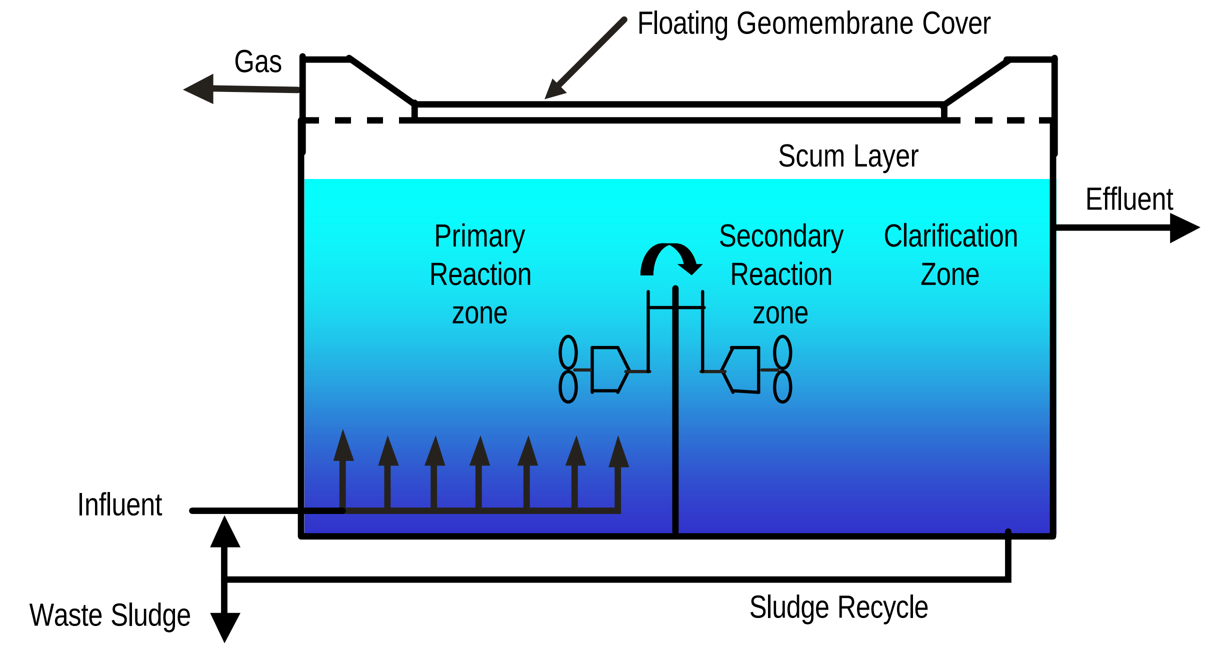
Expected Performance

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 50
TSS mg/L	130	70	< 50

Plant Footprint

IIWWTP: Approximately: 150,000 m^2 plus sludge handling
LGWWTP: Approximately: 38,000m^2 plus sludge handling

Schematics



Advantages

- Easier to operate and maintain than most high-rate anaerobic systems
- Low operating and maintenance costs
- Eliminates primary treatment because raw solids can be digested in the reactor
- Stable against shock loadings
- Equalization is built in because of the large volume
- Digests waste activated sludge
- Simplicity - no packing material, no special gas separation method, no moving parts, no mechanical mixing, and little plugging potential
- Ability to handle a wide range of waste characteristics including solids, oils and grease
- Simple and relatively economic construction
- Low loading and high effluent quality

Disadvantages

- Pumping and energy cost
- Large land area required
- Potential feed flow distribution inefficiencies
- Maintenance of the geomembrane cover

Comment

- Not suitable for low strength wastewater

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The BVF is sized for a hydraulic retention time of 7 days and for a maximum COD loading rate of 1 kg/m^3/day. Because of the large HRT, a large equalization basin is not required. However a smaller tank is required for mixing of different waste streams and for conditioning. The tank has several uses including:

- Mixing of chemicals for alkalinity control,
- Addition of nutrients,
- Mixing of influent with clarified effluent in order to blend batch slugs of FOG and septage, and
- Providing a constant flow to the heat exchanger where the heat from the effluent is transferred to the influent.

BVF can deal with wastewaters having high concentrations of suspended solids and FOG.

4.3.1.6 Hybrid Reactor Combining USAB And Fixed Film

The hybrid system generally consists of a tank with a UASB in the bottom section and a fixed film reactor above. The process schematics and technical facts are summarized in Figure 4.21. In the UASB reactor, a granular sludge develops and provides the first phase of treatment. In the upflow fixed film reactor above, further treatment is provided for enhanced COD removal. Typical uses of a hybrid system are for high COD low TSS wastewater where high removal efficiencies are required.

In general, the reactors are sized to provide sufficient hydraulic retention time as required for the anaerobic treatment. Solids separation units are necessary following the anaerobic reactors. Heat supply and reuse are also common in the anaerobic operation to improve the reaction rates. Off-gas control is essential to recover methane, burn off foul gases and odour prevention.

4.3.1.7 Proven Technology

Anaerobic processes are considered proven technology for sludge, high strength industrial wastewater, and recalcitrant substance pre-treatment. In most full-scale applications, the anaerobic processes are used to treat high concentration wastewater, typically with BOD greater than 2,000 mg/L in order to sustain the desired reaction rate. However, an anaerobic process is not commonly used to treat municipal sewage with normal domestic organic and solids concentrations (e.g. 200 mg/L of TSS and 200 mg/L of BOD_5).

4.3.1.8 Discharge Requirement/Effluent Quality

The organic removal efficiency of anaerobic processes is typically less than that of aerobic processes. Poor sludge settleability is commonly observed in anaerobic processes and results in high effluent TSS concentration using conventional gravity settling. Expected effluent quality is 50 mg/L of BOD_5 and 50 mg/L of TSS.

FIGURE 4.21
ANAEROBIC HYBRID REACTOR - UASB & FIXED FILM

Process Description

The anaerobic hybrid reactor is a combination of the Upflow Anaerobic Sludge Blanket (UASB) process and the Upflow Fixed Film (UFF) process. The UASB reactor forms the lowermost portion where a flocculant and / or granular sludge develops. The upper UFF reactor consists of a plastic media which provides an extensive surface area for the fixed-film biomass to grow. Wastewater enters the hybrid reactor through an influent distribution system and is mixed with recycle effluent from the bottom. It then enters the sludge bed and passes through the UFF section. The excess sludge is wasted on a regular basis through a header-lateral system on the bottom of the reactor. The performance of a hybrid reactor depends on its capacity to maintain high amounts of biomass inside the reactor.

Design Criteria

Influent TSS loading should not exceed 10 to 20% of the COD load
Fats, oils and grease concentrations should not exceed 100 to 200 mg/L
Temp: 30 - 37^{o}C
HRT: 16 - 36 hr
COD loading: 5 -10 kg/m^3.d

Expected Performance

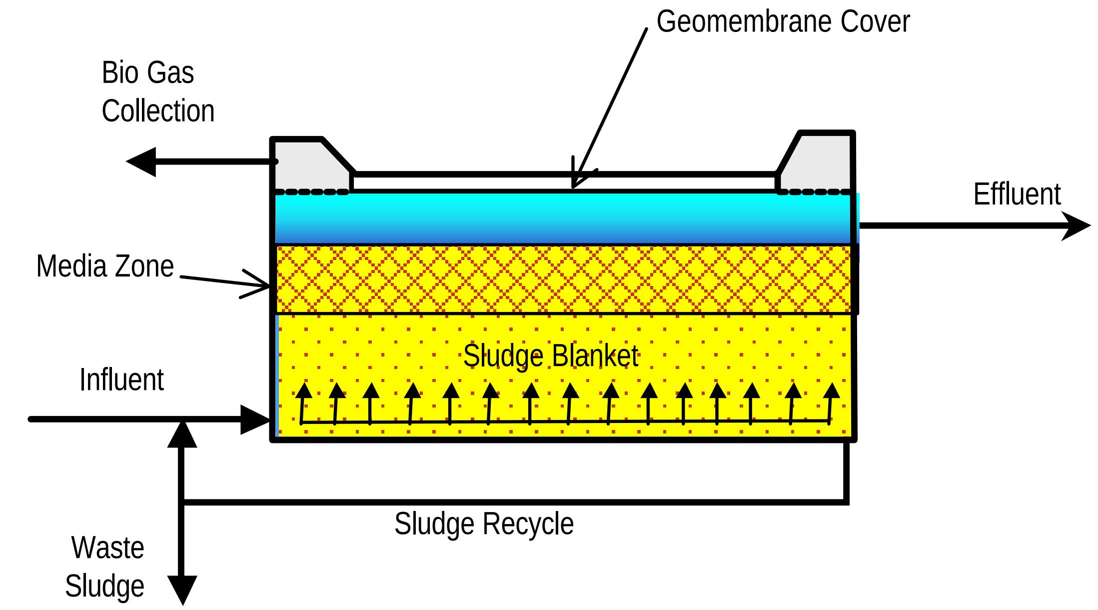
COD removals: 70% - 90%

Parameter	Raw Wastewater	Primary Effluent	Biological Treatment Effluent
BOD_5 mg/L	132	90	< 45
TSS mg/L	130	70	< 50

Plant Footprint

IIWWTP: Approximately: 260,000 m^2 plus sludge handling
LGWWTP: Approximately: 66,000m^2 plus sludge handling

Schematics



Advantages

- Does not need granular sludge
- Stable and resilient to shocks
- Produces better effluent that UASB reactors on chemical wastewater
- Superior for wastewater on low sludge yield
- Increased solid retention time promotes higher removals

Disadvantages

- Pilot testing is advisable for questions regarding impact of loadings on system
- Accumulation of non-attached biomass in the media resulted in channeling and dead zones
- Pumping and energy cost

Comment

- Suitable for high strength wastewater
- Not an economic option

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Ammonia removal may not be efficient because of ammonification and cell lysis in anaerobic condition. This is not an acceptable target for the build-out to secondary.

4.3.1.9 Reliability

Anaerobic process operation is highly dependent on the stability of operational conditions, including pH, temperature, mineral concentrations, and loading variances. It will take weeks even months to recover from any major system upset, thus the anaerobic process is not considered to be a reliable operation.

4.3.1.10 Site Suitability

The anaerobic process requires less footprint expansion than the aerobic process, and it is suitable at IIWWTP and LGWWTP sites. However, the anaerobic processes require significant expansion to construct the bioreactors, cover domes, clarifiers, pumping/piping for solids, gas handling, and corrosion protection, which demand much more capital investment than the aerobic process. Due to the nature of low-strength wastewater sources, as well as sophisticated operational control requirements, anaerobic processes are not recommended for either plant.

4.4 FIXED FILM SUSPENDED GROWTH

4.4.1 Trickling Filter/Activated Sludge

4.4.1.1 Process Description
(WEF MOP 8, 4th edition)

A summary description and process diagrams are provided in Figure 4.22. Dual or coupled biological treatment systems have a fixed film reactor and a suspended-growth process. This combination results in a two-stage unit process that has unique design parameters; its treatment efficiency capabilities often exceed the individual performance of both parent systems. The activated-sludge (suspended-growth) unit provides a variety of functions, including flocculation to improve clarification, removal of residual soluble 5-day biochemical oxygen demand (BOD_5), nitrification, denitrification, and phosphorus removal to meet advanced wastewater treatment (AWT) requirements.

a) High Rate Or Roughing Trickling Filter- Activated Sludge (RF/AS)

A common method of upgrading existing activated-sludge plants is to install a roughing filter ahead of the activated-sludge process. As part of the roughing filter/activated sludge (RF/AS) process, the roughing filter is typically 15 to 30% of the size required if treatment had been accomplished through the use of the trickling filter process alone. Hydraulic retention time in the aeration basin is typically 35 to 50% that required with the use of the activated-sludge process alone.

TRICKLING FILTER ACTIVATED SLUDGE (TF/AS)

FIGURE 4.22
TRICKLING FILTER ACTIVATED SLUDGE (TF/AS)

Process Description

(See also Trickling Filter - Standard Rate) The trickling filter process, using an upstream trickling filter followed by an activated-sludge process, is often used to upgrade an existing activated sludge system. The principal difference between this process and TF/SC is that the aeration basin is larger and that the suspended growth accomplishes a significant fraction of the total BOD removal. Trickling filter effluent is fed directly to the activated-sludge process without clarification and settled solids from the secondary clarifier are returned to the activated-sludge aeration basin. The most common application of this process is where the trickling filter is designed as a roughing filter for 40% to 70% of BOD removal.

Design Criteria

Trickling filter loading: 1.2 - 4.8 kg BOD/m^3.d
Activated sludge HRT: 2 - 4 hr
Activated Sludge SRT: 2.0 - 7.0 d
Activated Sludge MLSS: 2500 - 4000 mg/L
Clarifier peak overflow rate: 2.0 - 3.5 m/h

Expected Performance

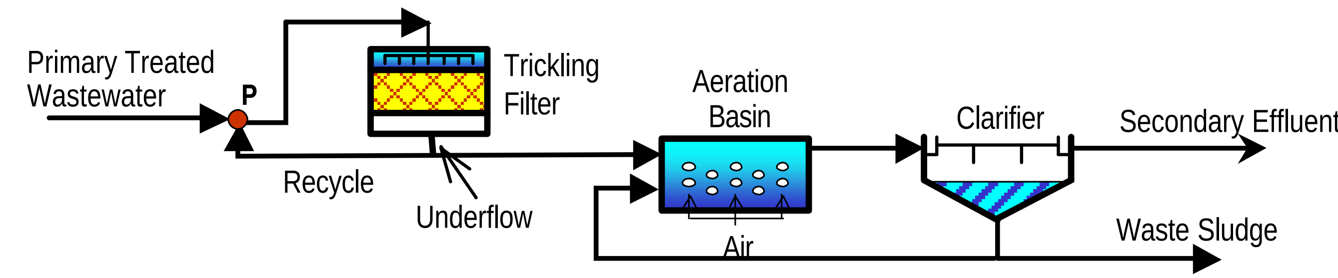
Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

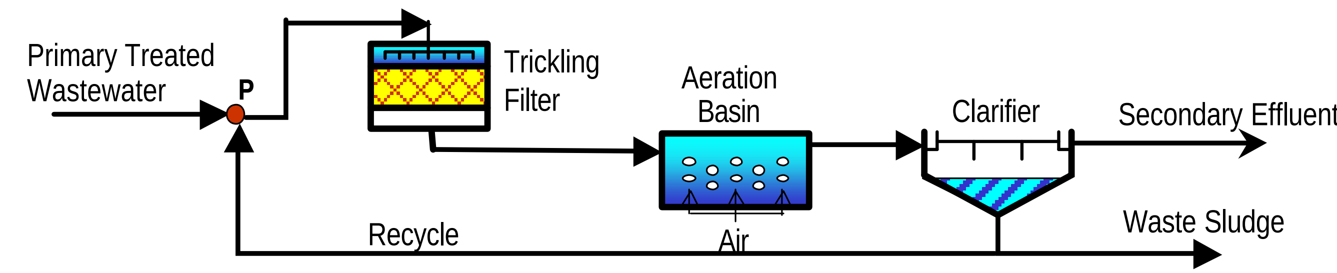
May fit on the Iona and Lions Gate sites

Schematics

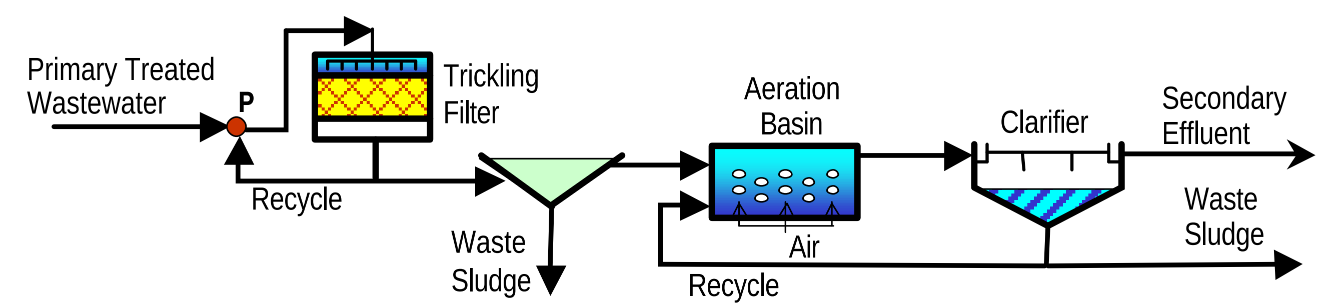
RF/AS



BF/AS



TF/AS
(Seldom
Used)



Advantages

- Proven Technology
- Simple operation
- Robust process resistant to toxic and hydraulic shocks
- Low energy requirement for aeration in Trickling Filter
- Entire biomass cannot be washed out by high peak flows
- Does not suffer from filamentous bacteria, sludge bulking, foaming
- Biomass has excellent settling qualities
- Smaller footprint than activated sludge

Disadvantages

- Possible psychoda (filterfly) nuisance
- Odour potential from trickling filter
- Can have High trickling filter profile (visual)

Comment

- RF/AS is the most likely of the three processes to be considered.

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Both TF/SC and RF/AS have the same process schematic, but with RF/AS, a much smaller TF is used so that the aeration basin must provide a significant amount of oxygen, BOD removal, and solids digestion. This differs from the TF/SC process where the TF is larger and provides almost all of the SBOD treatment, allowing the contact channel to provide only enhanced solids flocculation and effluent clarity. A deciding consideration in determining whether it is best to use the TF/SC or RF/AS process is often the availability of existing treatment units that influence the balance between capital and operating expenses.

b) Biofilter-Activated Sludge (BF/AS)

The biofilter-activated sludge (BF/AS) process is similar to that of RF/AS except that, with BF/AS, return activated sludge (RAS) is recycled over the fixed film reactor similar to the recycle of the BAF process. Incorporating RAS recycle over the fixed film reactor has sometimes reduced bulking from filamentous bacteria, especially with food-processing wastes, which are difficult to treat. Although it has sometimes improved solids settleability, there is no evidence that sludge recycle improves the oxygen-transfer capability of the biological filter.

a c) Trickling Filter-Activated Sludge (TF/AS)

The trickling filter-activated sludge (TF/AS) process is designed for high organic loads similar to those of RF/AS or BF/AS. However, a unique feature of TF/AS is that an intermediate clarifier is provided between the fixed film and suspended-growth reactors. The intermediate clarifier removes sloughed solids from the fixed film reactor before the underflow enters the suspended-growth reactor.

A benefit of using the TF/AS mode of combined process is that solids generated from CBOD removal can be separated from second-stage treatment. This is often a preferred mode where ammonia removal is required and the second stage of the process is designed to be dominated by nitrifying microorganisms. Another advantage in using intermediate clarification is reduced effects from sloughing of the fixed film on the suspended-growth portion of the plant. However, designers generally do not believe there is evidence of significant reduced oxygen demand or improved solids settleability from use of intermediate clarification. To eliminate the cost of intermediate clarification, most high-rate or roughing filters are designed as RF/AS or BF/AS, rather than in the TF/AS mode.

4.4.1.2 Proven Technology
(WEF MOP 8, 4th edition)

Technologies currently available can produce AWT effluents of 10 mg/L BOD_5 and TSS or less and ammonium-nitrogen effluents of 1 mg/L or less. Trickling filters have historically been considered vulnerable to climatic changes because wastewater droplets must be exposed to large volumes of ambient-temperature air. However, proper engineering design can reduce temperature losses caused by wind and ventilation to less than 1.5°C (2.7°F). Improving dosing procedures and minimizing recirculation can also help control temperature loss.

Temperature effects on nitrifying trickling filters are now considered to be less significant than those on activated. Earlier observations of poor effluent quality in winter were caused by a combination of shallow filters with high surface area, low freeboard, and high recirculation ratios that caused excessive heat losses. Other conditions contributing to poor performance included poor clarifier designs and filter dosing procedures that caused excess solids accumulations.

Trickling filters are no longer viewed only as a process to produce secondary treatment effluent. The TF process now used for AWT produces low residual BOD_5, TSS, and ammonium-nitrogen. Replacing existing TFs is often more expensive than updating and expanding existing units using known process technology such as the addition of short-term aeration or the solids-contact process. For higher loads the addition of activated sludge is practical.

In applications where more stringent effluent quality standards have exceeded the capability of existing TF designs, expanding TF capabilities often meets the requirements. Based on recent experiences, the full potential of the TF is only now being realized. The improved treatment capabilities of new and modified facilities, along with inherent ease of operation and low power use, have resulted in continued use of TFs.

4.4.1.3 Discharge Requirements/Effluent Quality

Regarding the influence of effluent quality on the choice among various combined processes, designers generally agree that combined processes can produce an effluent quality that is equal if not better than either of the activated-sludge or trickling filter parent processes. Effluent with less than 20 mg/L BOD_5 is typically achieved with good combined process design, and 10 mg/L BOD_5 has been achieved without advanced treatment at some facilities.

The RF/SC, BF/AS or TF/AS process will easily meet the LWMP requirements for treated effluents to the environment of 45 mg/L BOD_5 and 45 mg/L TSS maximum day allowable discharge. Should it be required, the TF reactors can be increased in height to provide ammonia removal through nitrification, but at increased cost.

4.4.1.4 Reliability (WEF MOP 8, 4th Edition)

Successful conventional secondary and AWT applications are achievable with TFs but require a better understanding of TF operation and required appurtenances. If proper design procedures are used, TF performance equaling that of suspended-growth systems can be achieved:

- Trickling filters can produce effluent qualities of < 10 mg/L BOD_5 and TSS;
- The effluent can be comparable to activated-sludge effluent;
- Trickling filters rapidly reduce soluble BOD_5 in applied wastewater;
- Temperature loss is less than 1.5°C in cold climates;

- Trickling filters are efficient nitrification units and effluents of < 1.0 mg/L ammonium-nitrogen can be produced;
- Natural ventilation is inadequate for optimizing performance and power ventilation should be used;
- For rotating arm applicators, trickling filters should be dosed every 10 to 60 seconds, but routine flushing; at 10 to 30 minutes/dose is also needed to enhance performance; alternatively, solid set and pumped application has wider flushing capability;
- Recirculation is typically beneficial for optimum performance, especially if the hydraulic loading rate is low;
- Power consumption is typically 25% less than activated-sludge treatment;
- Trickling filter sloughing cycles are harmful to filter performance and can be avoided by daily flushing; and
- Less land area is required for TFs than for activated-sludge treatment.

4.4.1.5 Site Suitability

Combined processes often require slightly less land than do other biological treatment processes for two reasons: the ability to construct tall fixed film reactors with heights of 4.9 to 9.8 m (16 to 32 ft) and the use of slightly higher loadings of both the fixed film and suspended-growth systems. However, saving space is typically not an overriding advantage in choosing combined processes because the savings tend to be insignificant or other processes can be modified to realize similar space savings.

The RF/SC, BF/AS or TF/AS facility is suited to either the Iona or the Lions Gate sites. For Iona, the site is of sufficient size to easily accommodate the TF. For the Lions Gate site the TF process could be added as a downstream process to the primary clarifiers. While the RF/SC, BF/AS or TF/AS process reduces site requirements over activated sludge alone, the TF/SC is likely more reasonable for the Lions Gate site.

4.4.2 Moving Bed Activated Sludge

4.4.2.1 Process Description

(Metcalf & Eddy, 2002)

There are now more than 10 and counting, different variations of processes in which a packing material of various types is suspended in the aeration tank of the activated sludge process. Typical examples of activated-sludge treatment processes with suspended packing include the Captor®, Linpor®, and Kaldnes®. Figure 4.23 shows the Kaldnes® process. Process schematics for Captor® and Linpor® are similar, except that there may be an internal recycle in the aeration basin to redistribute packing that accumulates at the reactor outlet.

FIGURE 4.23
KALDNES MOVING BED™ ACTIVATED SLUDGE

Process Description

The Kaldnes Moving Bed™ process utilizes biofilm carrier elements, which are designed to provide a large protected surface area for the biofilm and optimal conditions for the bacteria culture, to remove BOD, COD, ammonia and nitrate.

The Kaldnes carrier elements are small, neutrally buoyant plastic cylinders which are submerged in an aeration tank. Biomass grows on the media, which is trapped inside the bioreactor by an appropriately sized screen. A secondary clarifier is required to settle sloughed solids. The Kaldnes Moving Bed™ process can also be used as a roughing pretreatment ahead of activated sludge.

Design Criteria

HRT: 3 hours
Tank depth: 4 - 5m
Media Specific Surface: $500m^2/m^3$
Packing fills 25% to 50% of aeration volume
Final settling tank: surface overflow rate 18 $m^3/m^2 \cdot day$
solids loading rate 4.5 - 5.0 $kg/m^2 \cdot day$

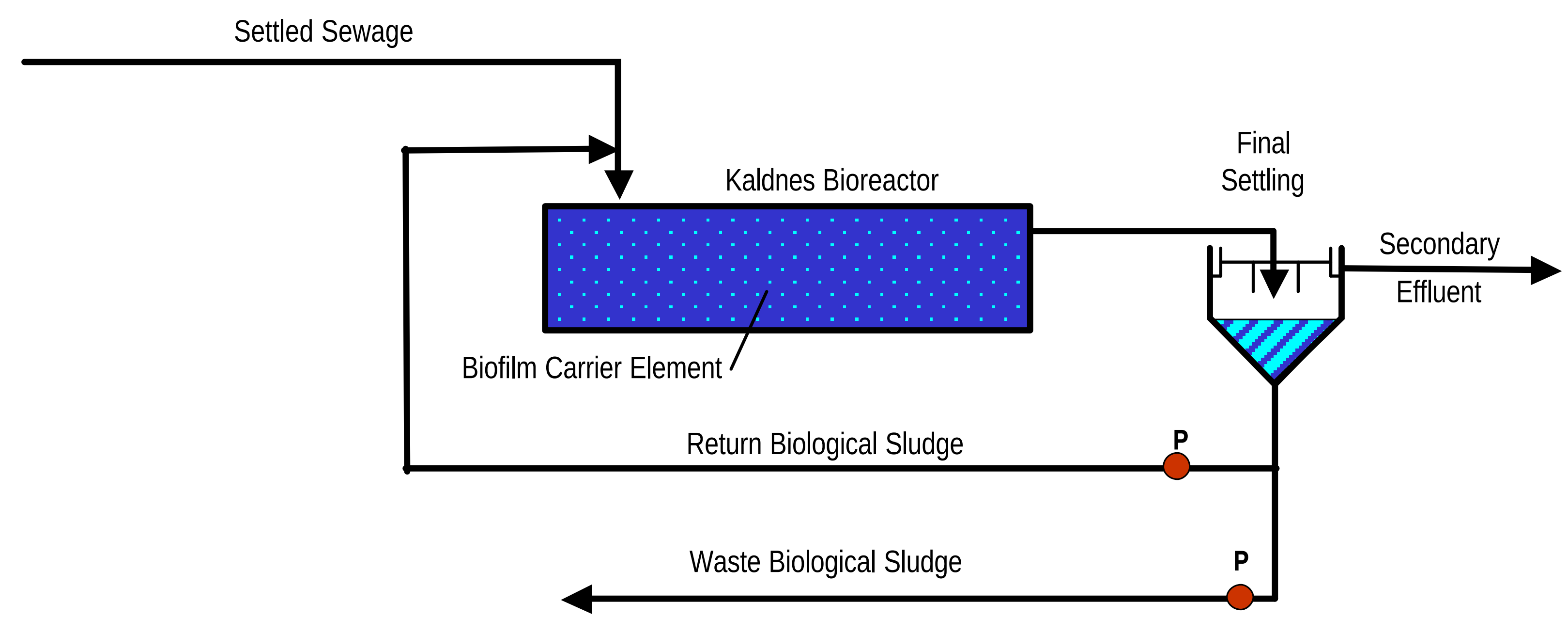
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Would impose additional land upon existing primary facilities, but less than conventional activated sludge. May be accommodated on the Lions Gate site.

Schematics



Advantages

- Higher Load Capability than CAS
- Less Susceptible to shock Load than CAS
- Smaller Sludge Volume than CAS
- LWMP Effluent goals Met - BOD, TSS
- Can Remove MBAS and ammonia toxicity

Disadvantages

- Limited Track Record
- Proprietary Process
- Different Process than Annacis / Lulu

Comment

- Possible option for Lions Gate - Cramped Site
- Could be applied at Iona depending on economics
- Most often used to upgrade existing plants

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a) Captor® and Linpor®

In the Captor® and Linpor® processes foam pads with a specific density of about 0.95 g/cm^3 are placed in the bioreactor in a free-floating fashion and retained by an effluent screen. The pad volume can account for 20 to 30 percent of the reactor volume. Mixing from the diffused aeration system circulates the foam pads in the system, but without additional mixing methods, they may tend to accumulate at the effluent end of the aeration basins and float at the surface. An air knife has been installed to continuously clean the screen and a pump is used to return the packing material to the influent end of the reactor. Solids are removed from a conventional secondary clarifier and wasting is from the return line as in the activated-sludge process.

The principal advantage for the sponge packing systems is the ability to increase the loading on an existing plant without increasing the solids load on existing secondary clarifiers, as most of the biomass is retained in the aeration basin. Loading rates for BOD of 1.5 to 4.0 kg/m^3/d with equivalent MLSS concentrations of 5,000 to 9,000 mg/L have been achieved with these processes. Based on the results of full-scale and pilot-scale tests with the sponge packing installed it appears that nitrification can occur at apparent lower SRT values, based on the suspended growth mixed liquor, than those for activated sludge without internal packing.

b) Kaldnes®

A moving-bed biofilm reactor (MBBR) has been developed by a Norwegian company, Kaldnes Miljøteknologi. The process consists of adding small cylindrical-shaped polyethylene carrier elements (specific density of 0.96 g/cm^3) in aerated or non-aerated basins to support biofilm growth. The small cylinders are about 10 mm in diameter and 7 mm in height with a cross inside the cylinder and longitudinal fins on the outside. The biofilm carriers are maintained in the reactor by the use of a perforated plate (5 x 25 mm slots) at the tank outlet. Air agitation or mixers are applied in a manner to continuously circulate the packing. The packing may fill 25 to 50 percent of the tank volume. The specific surface area of the packing is about 500 m^2/m^3 of bulk packing volume. The MBBR does not require any return activated-sludge flow or backwashing. A final clarifier is used to settle sloughed solids. The MBBR process provides an advantage for plant upgrading by reducing the solids loading on existing clarifiers. The presence of packing materials discourages the use of more efficient fine bubble aeration equipment, which would require periodic drainage of the aeration basin and removal of the packing for diffuser cleaning.

The most common design application is for BOD removal, nitrification, and denitrification. In a second type of application the MBBR is used in place of the trickling filter in the solids contact process.

4.4.2.2 Proven Technology

Not well commercialized in North America (three reported municipal installations, all designed for nitrogen removal). More common in Europe (numerous installations), where it is used for BOD removal and nitrogen removal.

4.4.2.3 Discharge Requirement/Effluent Quality

Submerged attached growth systems have been shown to meet 45/45 BOD/TSS secondary effluent standards (WEF MOP 8 pilot-scale data).

4.4.2.4 Reliability

The reliability in North America has not been well established.

4.4.2.5 Site Suitability

These are smaller foot print processes than activated sludge, and could be considered for Lions Gate. Depending on the proven economics use at Iona Island has not been ruled out.

4.4.3 Submerged Attached Growth

4.4.3.1 Process Description
(Metcalf & Eddy, 2002)

There are now more than half a dozen, and counting, different variations of processes in which a fixed packing material is placed in the aeration tank of the activated-sludge process. Three typical examples of fixed packing processes include the Ringlace® and BioMatrix® processes, Bio-2-Sludge® process, and submerged RBC's. A process schematic and summarized technical facts for Ringlace® are presented in Figure 4.24.

Ringlace® packing is a looped polyvinyl chloride material that is about 5 mm in diameter. It is placed in about 25 to 35% of the activated-sludge basin volume in modules with individual strands at 40 to 100 mm apart. The specific surface area provided ranges from 120 to 500 m^2/m^3 of tank volume. The packing placement location in the aeration tank is important. To provide efficient contact with the wastewater the packing should be placed along one side of the aeration vessel with the aeration equipment providing a spiral roll pattern for flow through the packing. Spiral roll aeration is usually less efficient than full floor coverage aeration with fine bubble diffusers.

FIGURE 4.24
SUBMERGED ATTACHED GROWTH (RINGLACE®)

Process Description

Primary effluent is discharged to an aeration basin containing the submerged Ringlace biomedia. The media consists of continuous strands of plastic fibers containing loops in a matrix form mounted on a rack. The biomass develops in a slime layer attached to the media. Oxygen is supplied to maintain an aerobic condition. The microorganisms (biomass) break down the organics in the wastewater and enable BOD removal and nitrification.

Design Criteria

HRT: 4 hours
Specific surface of media: 120 to 500 m^2/m^3 tank volume
Media: 25% to 35% of activated sludge basin volume

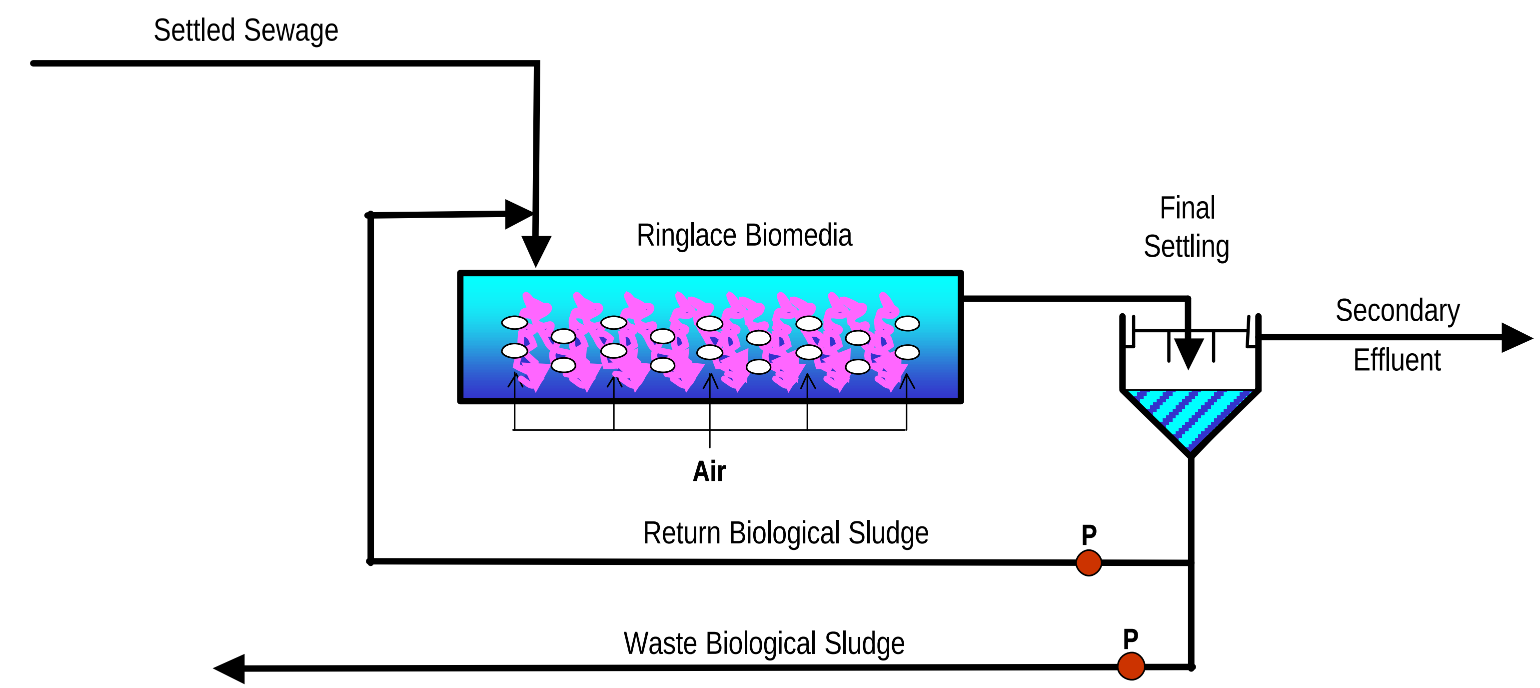
Expected Performance

Parameter	Percent Removal
BOD_5 mg/L	> 90%
TSS mg/L	> 90%

Plant Footprint

Would impose additional land upon existing primary facilities, but less than conventional activated sludge. May be accommodated on the Lions Gate site.

Schematics



Advantages

- Higher Load Capability than CAS
- Less Susceptible to Shock Load than CAS
- Smaller Sludge Volume than CAS
- LWMP Effluent Goals Met - BOD, TSS
- Can Remove MBAS and ammonia toxicity

Disadvantages

- Different Process than Annacis / Lulu
- Limited Track Record, mainly small-scale
- Microbiological Problems - Worms
- Proprietary Process

Comment

- Possible option for Lions Gate - Cramped Site
- Could be applied at Iona depending on economics

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The location along the length of the tank is also important for nitrification and denitrification system operations. A location is recommended where sufficient BOD remains to develop a biofilm growth, but where the BOD demand is low enough so that ammonia oxidation can occur in the film. The optimal rate can be difficult to achieve as variations in BOD loading can vary the biofilm growth on the packing and the competition between heterotrophic and autotrophic bacteria for surface area. In some applications, the advantages of using a fixed internal packing can be negated due to the growth of bristle worms in the biofilm.

4.4.3.2 Proven Technology

Not well commercialized in North America (two reported municipal applications). Most applications are in Japan, reportedly for lightly loaded small-scale installations (WEF MOP 8).

4.4.3.3 Discharge Requirement/Effluent Quality

Has been shown to meet 45/45 BOD/TSS secondary effluent standards. Parallel testing of Ringlace®/activated sludge with activated sludge alone at the University of B.C. Civil Engineering BNR pilot-scale facility did not demonstrate any improvements in performance by addition of Ringlace® media.

4.4.3.4 Reliability

The reliability in North America has not been well established. Testing at the University of B.C. Civil Engineering pilot plant resulted in extensive infestations of worms, causing maintenance problems.

4.4.3.5 Site Suitability

These are smaller foot print processes than activated sludge, and could be considered for Lions Gate. Depending on the proven economics use at Iona Island has not been ruled out.

4.5 MISCELLANEOUS

4.5.1 Advanced Oxidation

4.5.1.1 Process Description

Advanced Oxidation includes a variety of processes which utilize ozone, hydrogen peroxide, hypochlorite and other powerful oxidants to destroy BOD in wastewater, either partially or fully.

Ozone and or peroxide are sometimes used in association with UV. Hydroxyl radicals are produced and these radicals react extremely rapidly with BOD and organisms reducing both the BOD and the number of organisms in the effluent.

Electrolysis, which can also produce hydroxyl radicals, is used as part of an electro-flocculation process for the removal of suspended materials. A process schematic and summarized technical facts for advanced oxidation are presented in Figure 4.25.

This recently applied technology is developing rapidly and has the potential to revolutionize wastewater treatment. One of the most advanced applications has been developed by Hydroxyl and named the Hydroxyl-UVO process. Another process known to the authors has recently been shown to produce an effluent complying with secondary treatment standards in a full -scale module for a small municipality at operating costs lower than conventional treatment and on a much smaller site.

Because this technology is still in the early stages of commercialization it is not suggested for inclusion in the processes considered at this stage. However the state of development should be evaluated before detailed design commences to establish whether it can then meet the requirements for consideration.

4.5.1.2 Proven Technology

Not well commercialized, still under development.

4.5.1.3 Discharge Requirement/Effluent Quality

Has been shown to meet 45/45 BOD/TSS secondary effluent standards.

4.5.1.4 Reliability

The reliability has not been well established.

4.5.1.5 Site Suitability

These are small foot print processes, which could be considered for Lions Gate. Depending on the proven economics use at Iona Island has not been ruled out.

FIGURE 4.25
ADVANCED OXIDATION (AOP)

Process Description

Advanced oxidation processes are generally based on the generation of the hydroxyl radical (HO•), which is a powerful chemical oxidant. The HO^- radical reacts with dissolved constituents in the wastewater. AO processes are normally used to oxidize complex organic molecules into relatively benign compounds that are easily biodegradable. The HO^- radical can be generated in a variety of ways, using chemical reactions that involve ozone, UV light, and hydrogen peroxide. The HO^- radical is very reactive and has an extremely short half-life.

Design Criteria

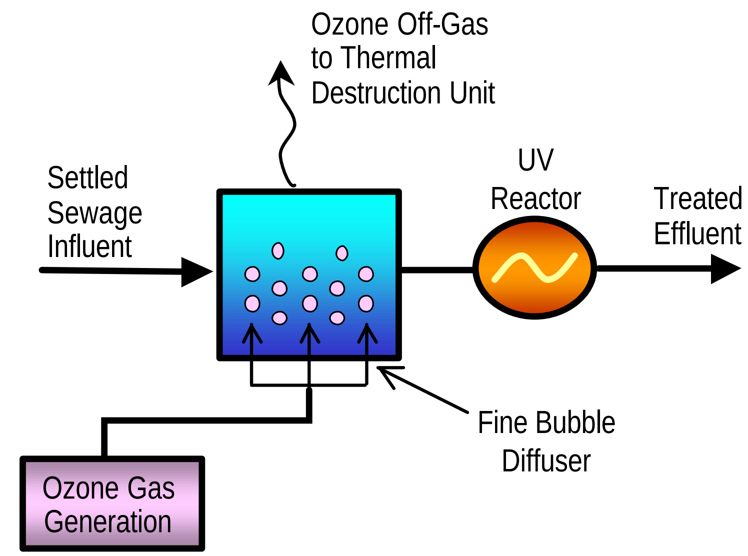
Ozone Electrical Energy Input: 21 to 265 kWh/log order reduction/1000m^3 with 5 to 6 mg/L dose of H_2O_2.

Expected Performance

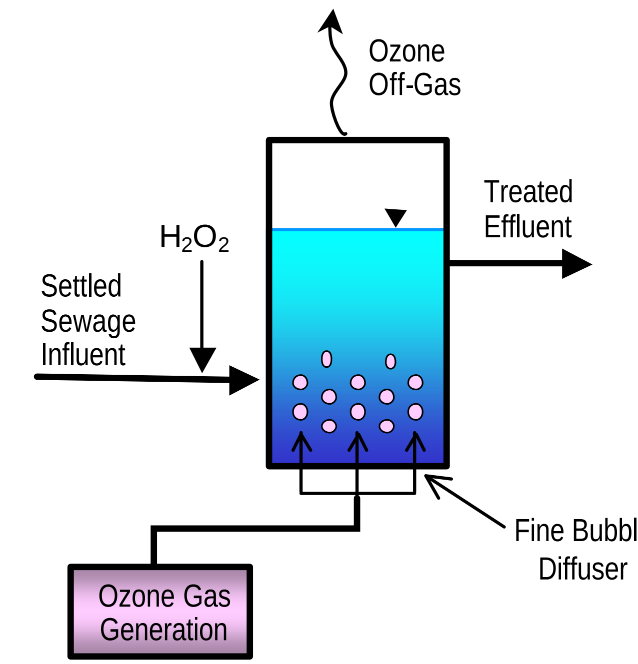
Plant Footprint

Should be accommodated on the Iona and Lions Gate sites

Schematics



OZONE/UV



OZONE/H_2O_2

Advantages

- Small plant footprint
- Wastewater constituents are degraded rather than concentrated or transferred into a different phase
- No need to dispose of secondary waste materials

Disadvantages

- High cost to generate hydroxyl radical
- High concentrations of carbonate and bicarbonate in some wastewater can react with the hydroxyl radical and reduce efficiency of the process
- Other factors such as SS, pH, type and nature of the residual TOC, and other wastewater constituents can also affect the process
- Pilot testing is almost always required to test the technical feasibility

Comment

- Usually apply to low COD wastewater because of cost. Mainly applicable to complex non-biodegradable wastes

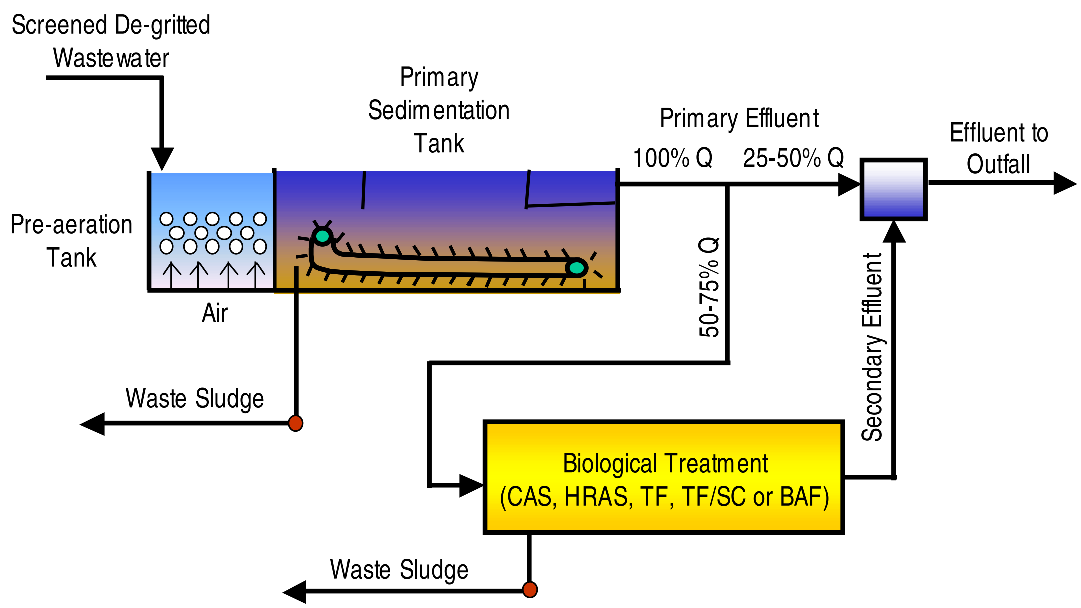
4.6 PRIMARY TREATMENT FOLLOWED BY PARTIAL BIOLOGICAL TREATMENT

The partial biological treatment option consists of providing primary treatment to 100% of the flow using the existing primary clarifiers. This is followed by a biological process, as described in Section 4.1 to Section 4.5 to treat only a portion of the primary effluent. The treated primary and secondary effluents would then be combined prior to discharge to the ocean. A general process schematic is illustrated in Figure 4.26. The portion of flow receiving biological treatment (e.g. 50 ~ 75%) depends on how much dissolved BOD must be removed in order to achieve secondary effluent quality objectives.

The design capacity of the biological process is less than 100% of the design flow. By biologically treating only a portion of the primary effluent, the savings in capital investment will be significant. For example, designing a secondary system to treat 1.5 times the ADWF, 75% of the primary effluent, will result in a 25% saving in capital cost and real estate compared with a secondary treatment plant sized for the entire design flow.

This arrangement is a logical option for IIWWTP, where, because of the combined sewer system, dry weather conditions require higher treatment capacity than wet weather conditions. At this plant, the peaking factor for wet weather flow is unusually high. During the dry weather season, 100% of the primary effluent can be treated in the secondary biological process. Part of the flow will bypass the secondary treatment system only during wet weather season when the flow exceeds the design capacity. The smaller footprint is another advantage of partial biological treatment.

FIGURE 4.26
PARTIAL BIOLOGICAL TREATMENT - GENERAL



4.7 CEP WITH PARTIAL BIOLOGICAL TREATMENT

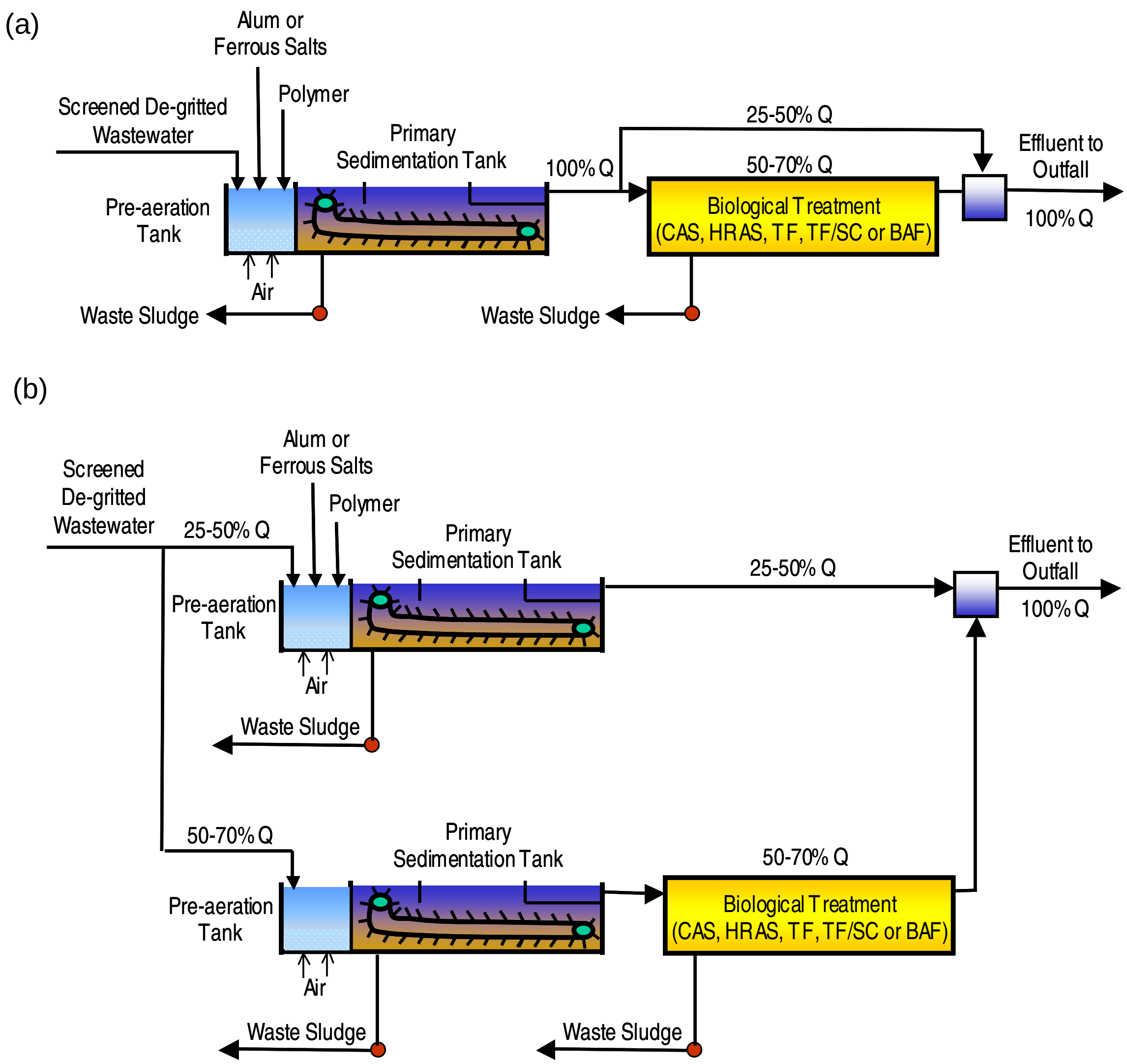
The combination of chemically enhanced primary (CEP and its modifications) and partial biological treatment (Section 4.1 to Section 4.5) can be considered as a secondary build-out option to achieve BOD_5 and TSS targets, as well as effluent toxicity reduction. Two possible process schematics are illustrated in Figure 4.27.

As shown in Fig. 4.27(a) and 4.27(b), 100% of the flow receives CEP. 50 – 70% of the primary effluent receives biological treatment. The remainder is bypassed around the biological reactor. The primary and secondary treated effluents are combined before prior to being discharged.

The CEP process will improve the TSS and BOD_5 removal efficiency in the primary treatment. A portion of the primary effluent can be treated in the biological process to achieve additional BOD_5 removal, and sequentially TSS removal in the secondary clarifiers. Operational conditions of CEP and biological treatment should be designed and controlled to accommodate the system requirements, including chemical dosage in CEP, F/M ratio, SRT, and aeration in the biological process. CEP can be operated on a continuous or intermittent basis (e.g. during peak loads and dry weather). However, the biological system must be operated continuously to maintain the stability of the system. CEP can also be operated to meet the requirements of the biological system (e.g. organic loads) and also effluent quality requirements (i.e. TSS and BOD_5 concentrations).

As in the case of primary treatment followed by partial biological treatment described in Section 4.6, significant savings of capital cost and land space can be achieved. In this option, the required biological process capacity will be lower than the option described in Section 4.6 due to the additional TSS and BOD removal by CEP. However, sludge production is expected to be higher than the Section 4.6 option. This is mainly due to chemical sludge generation and the capture of additional solids. The additional cost of sludge handling capacity and chemicals for operation may not justify the biological treatment operational cost saving (i.e. aeration energy).

FIGURE 4.27
CEP WITH PARTICAL BIOLOGICAL TREATMENT SCHEMATICS



5 SITE CONSTRAINTS AT LIONS GATE PLANT

5.1 SITE CONSTRAINTS AT LIONS GATE PLANT

Land Use and WWTP Location

The plant is located on small area of land leased from the Province of British Columbia and from the Vancouver Port Authority as shown on Figure 5.1. This land bounded in the north by the B.C. Rail tracks, the Burrard Inlet in the south, and Department of Highways Lions Gate Bridge ROW in the east. The present leased land area is approximately 3.8 ha.

Site Access

The present site access is over the BC rail tracks. This access is often closed due to the train marshalling activities of BC rail, but these interruptions are not unreasonable and BC rail attempts to minimize delays.

Water Table Ocean Levels

Much of the existing ground floor building elevations is at 4.45 m elevation (106 ft GVSDD datum), which is 2 m above the extreme HHW level in Burrard Inlet. Almost all buried tanks are subject to hydrostatic uplift pressures due to the high water table. New construction and design will have to accommodate the high water table issue.

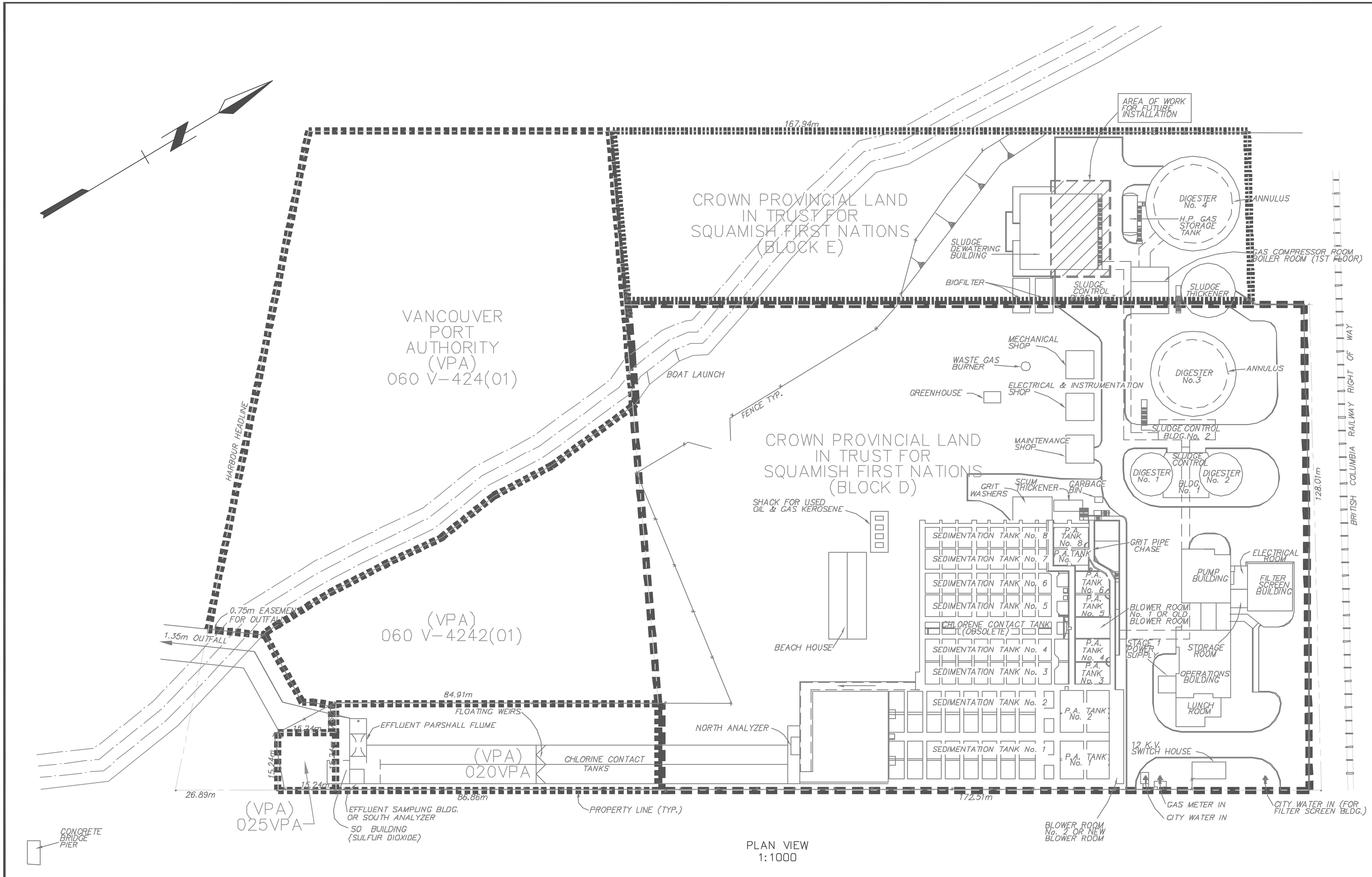
Geotechnical Issues

The salient geotechnical issues are the fact that the site subsurface soil conditions are prone to long term settlements and liquefaction during earthquake events. Post-liquefaction ground settlement is in the range of 0 to 250 mm. This settlement would not be uniform across the site. For preliminary assessment, half of the above noted settlement (125 mm) can be taken as differential over a distance of 5 m. The latter can be mitigated for new works by preloading and piling foundation designs and designing for the anticipated settlements. The liquefaction issue can not be completely addressed by any amount of site preparation measures, simply because the firm sand stone and shale ground is approximately 82 meters below grade, and the 82 meter thick top layer of deltaic sediments are prone to liquefaction. Ground improvements to mitigate these conditions are discussed in Appendix 9 and include a 15 m wide densified berm along the south and south-west boundary of the site.

Odour

The plant is located approximately 600 meters from the Park Royal shopping centre and the a RV park facility. As such odours are a concern. These can be minimized by odour treatment.

FIGURE 5.1 LIONS GATE WASTEWATER TREATMENT PLANT - LAND OWNERSHIP



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Visual Impact

The plant is visible from the shipping lane along which cruise ships access the harbour and from the vantage points in Stanley Park. Construction of tall structures could impose on the skyline as viewed from the housing development along the mountains of the North shore.

5.2 SECONDARY TREATMENT ON EXISTING SITE

By selecting appropriate secondary technology, the construction of treatment capacity to serve the population well beyond the year 2046 has been shown to be feasible (see Section 10).

The option of constructing the secondary expansion on the existing site would have significant advantage, since the plant already exists. The development of an alternative site as well as for the development of three separate sites have been assessed and are presented below.

5.3 ALTERNATIVE SITE FOR A SECONDARY TREATMENT PLANT

Introduction

It would be feasible to construct a new treatment plant to the east of the existing Lions Gate WWTP. LGWWTP, with the exception of the influent pump station, could then be demolished and the site made available for other uses. A block flow diagram of the proposal is presented in Figure 5.2.

Wastewater would be collected at the existing LGWWTP pump station and pumped through a new pipeline to a new treatment plant located within a radius of 1.5 km from the existing plant. Most of the wastewater originating in North Vancouver would gravitate to this proposed plant. Treated effluent would be pumped back from the WWTP on the new site to the existing LGWWTP site where it would be discharged to the Burrard Inlet through the existing outfall. Flows and plant sizing are based on a 2046 design horizon.

Forcemains

An allowance has been made for the construction of two new forcemains, one to convey untreated wastewater from the existing LGWWTP pump station to a new WWTP site, and the other to convey treated wastewater from the proposed treatment plant to the existing outfall. Details are set out in Table 5.1:

FIGURE 5.2 LIONS GATE WASTEWATER TREATMENT PLANT
BLOCK FLOW DIAGRAM - EXAMPLE ALTERNATIVE WWTP (2046)

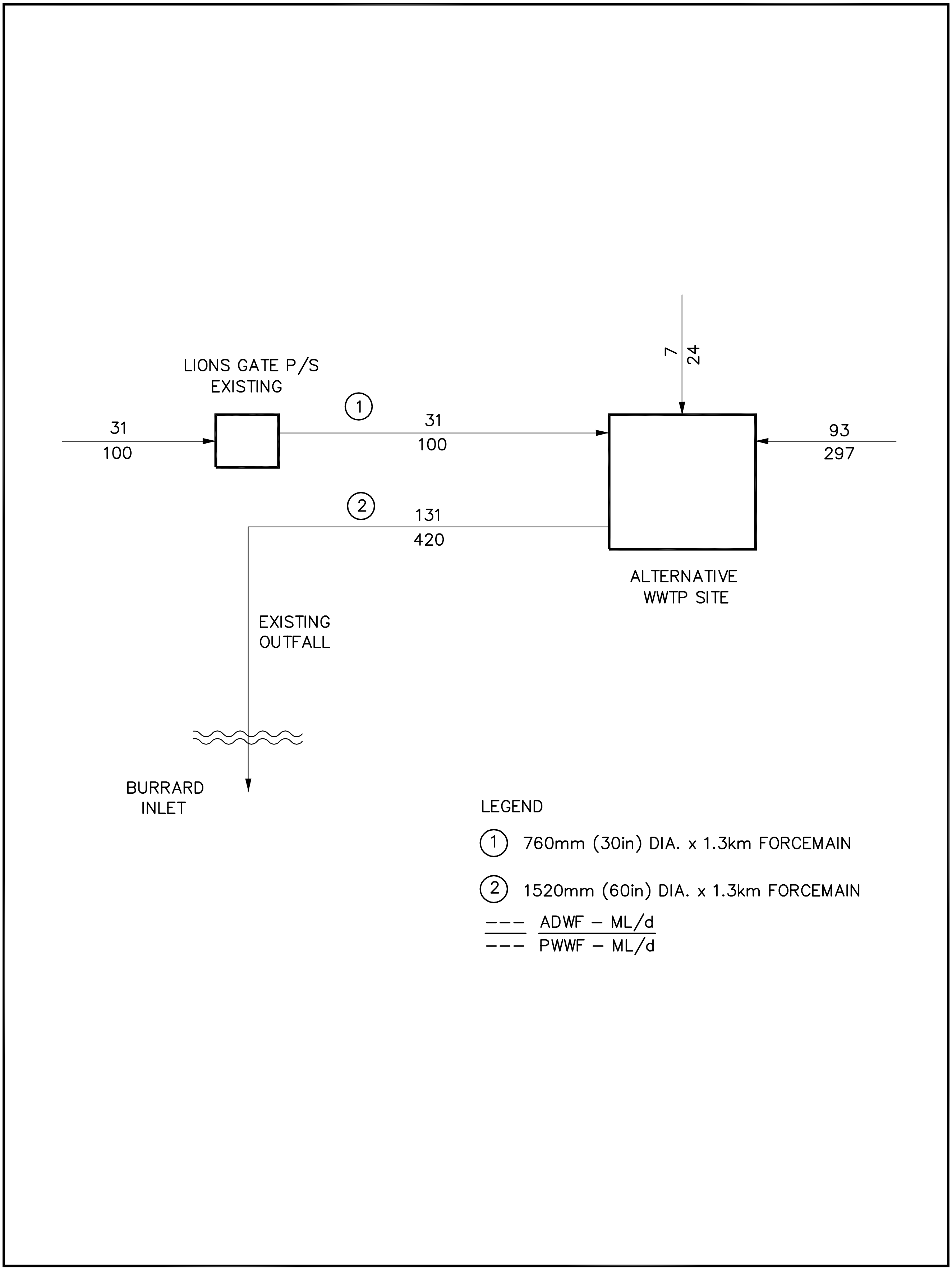

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TABLE 5.1
FORCEMAIN REQUIREMENT DETAILS

Description	Length (m)	Flow (ML/d)	Diameter (mm)	Est. Cost. ($million)
Influent Forcemain	1,300	101	760 (30 in.)	0.8
Effluent Forcemain	1,300	420	1,520 (60 in.)	1.8
Total	-	-	-	**2.6**

Routing has not been determined in any detail, although it is likely that each pipeline would be laid in a common trench along a route parallel to the existing railway line.

Pump Stations

The estimated 100 ML/d PWWF from LGWWTP to a new WWTP site is a fraction of the existing influent pump capacity. No allowance has therefore been made for pump station improvements.

The new Pump Station would include pumps capable of pumping 420 ML/d at a TDH of 9 m. The estimated construction cost is $2.8 million. Allowing 45% for redundancy, engineering and contingencies, the estimated total cost is $4 million.

Treatment Plant

The new WWTP would be constructed on a site within 1.5 km of the existing plant. The estimated land area is 6.9 Ha. Land ownership has not been addressed at this stage. Preliminary cost estimates for the total project are detailed below.

TABLE 5.2
PRELIMINARY COST ESTIMATES FOR EXAMPLE WELCH STREET WWTP

Description	Flow (ML/d)	Area (Ha)	Construct. Cost $\$10^6$	Total Cost $\$10^6$
New WWTP	131 (ADWF)	6.9	109	153
New P/S	420(PW WF)	Inc.	3	4
Forcemains	-	-	2	3
Totals	**-**	**6.9**	**114**	**160**

Constructions cost estimates are based on the consulting team's cost data. Total Costs are estimated to be Construction Costs x 1.4 and are inclusive of additional items such as noise control, earthquake protection, odour control, land purchase, architectural finishes, outfall, contingencies, engineering, financing and administration. Estimates are based on an ENR Index of 6794 (November 2003).

From Section 8.2 the total estimated cost of upgrading to an equivalent level of treatment on the existing LGWWTP site is approximately $100 million.

5.4 DISPERSED SECONDARY TREATMENT

Introduction

Real estate available for the future development of LGWWTP is limited. An alternative approach would be to carry out treatment at three dispersed sites, the existing LGWWTP and at two other plants. The costs and benefits of this strategy are briefly reviewed in this section.

Plant sizing is based on projected flows for the year 2046.

Information contained in the report "Computer Simulation Model Development North Shore Sewerage Area (NSSA) Stage 1: Runstdy Model"; September 1996, Reid Crowther, was used as a guide to relative flows within the North Shore System.

Treatment Plant Site Locations and Sizing

Figure 5.3 is a block flow diagram showing the flows to the three proposed plants. Figure 5.4 shows the example locations of the plants.

West Vancouver Waste Water Treatment Plant

The design ADWF would be 26 ML/d. The plant would be located in West Vancouver. Treated wastewater would be discharged into Burrard Inlet through a new outfall.

Lions Gate Waste Water Treatment Plant

This plant would be located at the existing LGWWTP site and would use the existing outfall and infrastructure on the treatment plant site. Secondary treatment would be designed for 66 ML/d ADWF.

FIGURE 5.3 LIONS GATE WASTEWATER TREATMENT PLANT

LOCATION PLAN - WELCH STREET WWTP

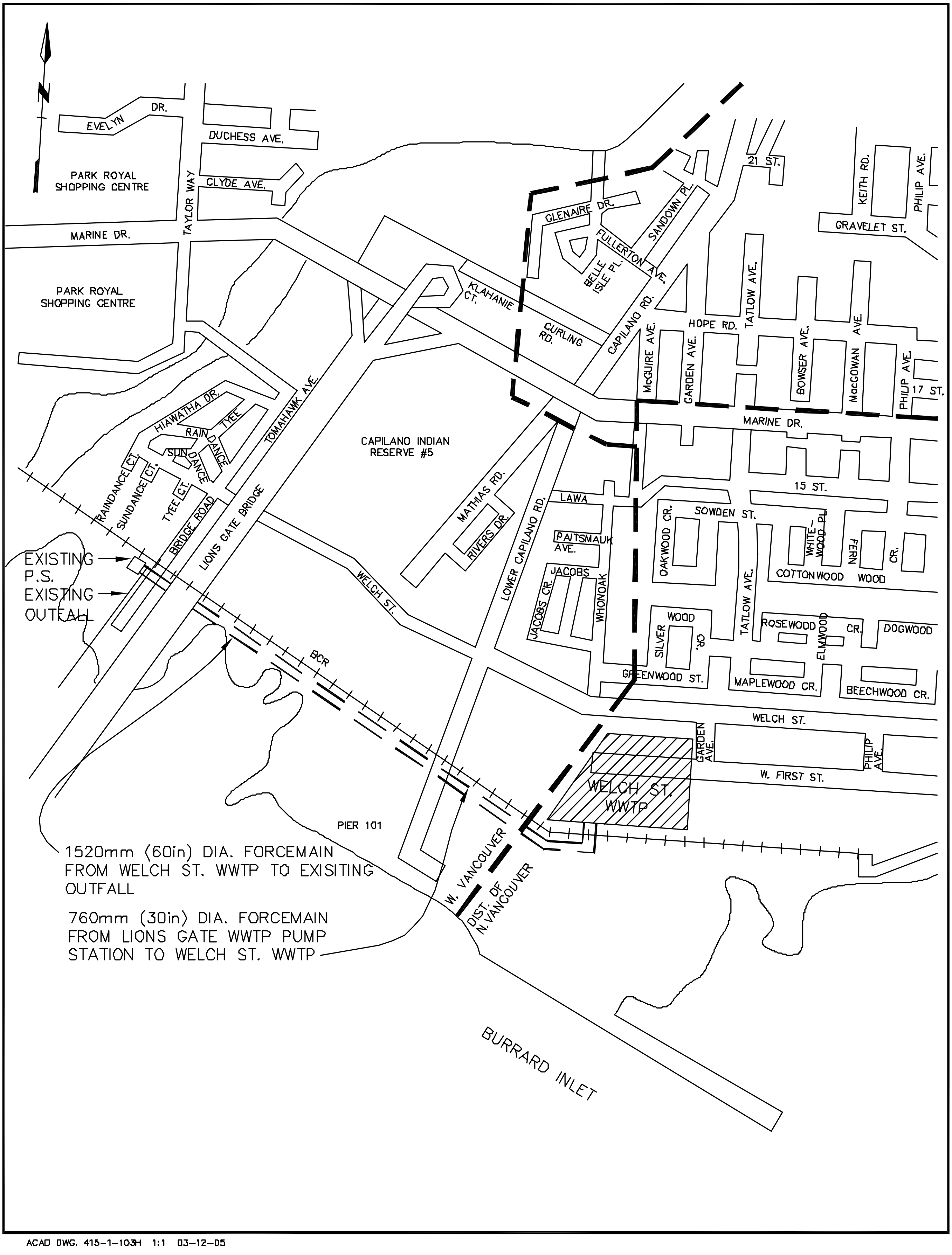

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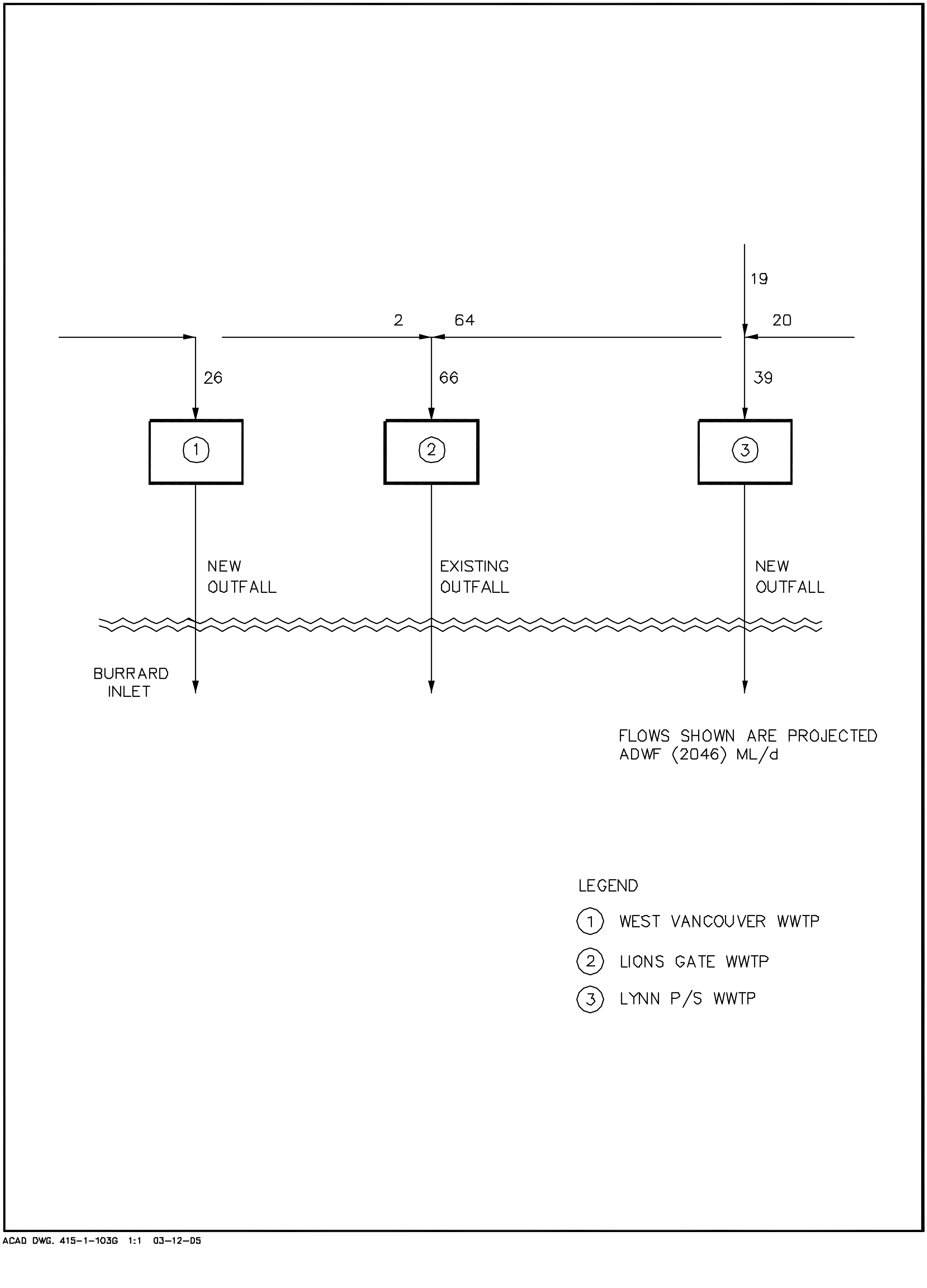
FIGURE 5.4 LIONS GATE WASTEWATER TREATMENT PLANT
BLOCK FLOW DIAGRAM - DISPERSED TREATMENT (2046)



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Lynn Pump Station Waste Water Treatment Plant

This plant would be located in an industrial zone near the existing Lynn Pump Station and designed for 39 ML/d ADWF. Discharge would be into Burrard Inlet upstream of the Lions Gate Bridge. Consequently, treatment would include biological nutrient removal.

Credit for Existing Sewers

As the wastewater would be distributed to three treatment plants, it would not be necessary to upgrade some North Shore trunk sewers that would to be upgraded if all flows were directed to LGWWTP or to a single replacement site. A credit of $5 millions has been allowed for twinning the North Vancouver City Section trunk sewer. For estimating purposes this has been assumed to be a 915 mm (36 in.) diameter sewer with a length of 7.5 km.

TABLE 5.3
PRELIMINARY COST ESTIMATES FOR DISPERSED SECONDARY TREATMENT

Treatment Plant	ADWF (ML/d)	Area (Ha)	Construct. Cost $\$10^6$	Total Cost $\$10^6$
West Vancouver inc. outfall	26	1.8	38	53
Lions Gate	66	3.4	66*	92*
Deduction for existing infrastructure			(27)	(38)
Lynn P/S inc. outfall	39	2.9	56	78
Totals	**131**	**8.1**	**133**	**185**

*: Greenfields construction cost

Construction cost estimates are based on D&K cost data. Total Costs are estimated to be Land Costs plus Construction Costs x 1.4 and are inclusive of additional items such as noise control, earthquake protection, odour control, land purchase, architectural finishes, outfall, contingencies, engineering, financing and administration. Estimates are based on an ENR Index of 6794 (November 2003).

The total project cost of a single new 131 ML/d plant near the existing plant is estimated to be $196 millions. The premium on the capital cost for dispersed treatment would therefore be approximately 8%. This small premium results from the credit taken for existing infrastructure at the Lions Gate site.

O&M costs for dispersed treatment would be higher than for a single treatment plant. The cost of power and chemicals would be approximately equal. However, additional manpower resources would be required, particularly as the Lynn P/S plant could be a BNR plant, which would require a higher level of control. Monitoring costs for the three plants would be higher. Annual plant maintenance costs would also be higher.

Sludge Treatment

Lynn Pump Station WWTP and West Vancouver WWTP would probably not include sludge digestion facilities, as use would be made of the digesters at the Lions Gate WWTP. Sludge would be conveyed to the plant using existing sewers and would settle out in the existing primary tanks.

Environmental Impact

Since the treatment plants would be designed to similar discharge standards and use similar amounts of consumables the environmental impact is likely to be the same.

Social Impact

The creation of an additional two treatment plant sites would impact on a greater number of residents. These impacts would include noise, odour, traffic and aesthetics. The process of procuring the necessary sites is likely to be protracted.

5.4.1 Discussion of Dispersed Treatment

Given the existence of a trunk sewer system delivering to the LGWWTP site, the creation of a dispersed secondary treatment system has little advantage to offer. The following disadvantages have been identified:

- Difficulty of acquiring land,
- Higher project cost,
- Higher operating and maintenance cost,
- More monitoring and administration,
- Increased social impact.

GVRD has advised that the cost of land for two new treatment plants is estimated to be in the order of $30 million.

6 SITE CONSTRAINTS AT IONA ISLAND PLANT

The initial impression is that space is not a concern at IIWWTP, as the east and west portions of the property could be used for the expansion of the facility. However, further analysis reveals that the following constraints must be taken into account.

Sludge Stockpiles

The east portion of the site is low and has been used to stockpile dewatered sludge for over 30 years. The sludge stockpiles will have to be relocated prior to proceeding with site preparation for expansion for some of the interim upgrade options and for all options related to built-out to secondary. GVRD has indicated that sludge stockpiles will be relocated as required to accommodate plant expansion.

Fill and Pre-loading

The east portion of the site will require the placement of about 4.5 m of fill in order to raise the site and prevent flooding. In conjunction with placing fill, the site must be preloaded for a period of at least one year. It should be noted that the existing plant site was preloaded prior to original construction over a 2-year period from 1959 to 1961. The plant expansion will also have to be preloaded prior to construction. However, in order to prevent settlement below existing structures, the pre-loading must be located at least 15 m from them.

As a result of pre-loading setbacks, it appears that additional digesters will have to be located east or south of the existing plant instead of west of the plant adjacent to the four existing digesters. Placing pre-loading west of plant could cause settlement of the berm around the sludge lagoon as well as under the effluent pump station, the maintenance building, the sludge thickener and the digesters. The requirements for pre-loading are discussed in more detail in the report by Trow Associates (Appendix 9).

Seismic Consideration

When subjected to an earthquake, the 15 meter thick layer of loose sand and gravel which underlay the site will behave like a heavy liquid. This will result in part-liquefaction consolidation settlement, loss of foundation bearing capacity and lateral spreading of the ground. To prevent lateral spreading, ground densification around the perimeter of the entire Iona Island treatment plant is proposed at an estimated cost of $1.7 million. Ground densification would consist of stone column (vibro-replacement) to 13 to 14 m depth on a triangular grid pattern at 2.8 m center-to-center spacing over a width of 15 m. To prevent vertical movement at existing structures, soil anchors would be required.

For any new addition, it is recommended that the footprint plus a 5 to 10 m wide area around the perimeter be densified. The shores of the Fraser River have a high value for habitat (high productivity and diversity). The design of the ground densification will have to take into account environmental protection of the shoreline.

Wetland

There are approximately 21 ha of land on the GVS&DD Iona Island WWTP property which is located east of the existing plant. Approximately half of this property is covered by wetland. Any new structures that extend into the south half of the property may encroach on existing wetland. New structures and tanks will be located on the north half of the site in order to minimize impact on the wetland. In order to minimize impacts on the wetlands, it is proposed to expand the plant on the north half of the property. However, if encroachment on the wetlands is necessary in order to accommodate the expansion, it may be necessary to provide some form of compensation for the loss of wetlands. This compensation could be a financial or by creating additional habitat in another location.

GVRD Parks

GVRD Parks has a maintenance yard located on the GVS&DD Iona Island WWTP property. The maintenance yard is located south-east of the primary clarifiers. Depending on the footprint of the expanded plant, this maintenance yard may have to be relocated. GVRD Parks has advised that they would like to have a 50 to 75 m wide strip on the south side of the sewage treatment plant property along McDonald Slough. Also GVRD Parks has advised they would like to maintain the gravel road located north of the property as this would allow access to the Parks property located east of the Iona Island WWTP property. It appears that the proposed upgrade will not impact the Parks property located east and south of the treatment plant site. Since no construction is proposed on most of the south half of the treatment plant property where the wetland is located, it appears that adding a 50 to 75 m strip of land to the Park property will be possible.

Shoreline of the Fraser River

There is an existing gravel road along the north side of the sewage treatment plant property along the Fraser River. The distance from the gravel road to the river is about 75 metres. The area south of the gravel road has been disturbed by sludge/biosolids storage while the area north the gravel road has been left in its natural state. It is proposed to locate any expansion of the treatment plant on land currently used for sludge storage and not to disturb the area between the gravel road and the shore of the Fraser River.

7 FIRST LEVEL OF SCREENING AND RANKING

7.1 DESCRIPTION OF SCREENING PROCEDURE

Details of the screening procedure are set out in Appendix 3 Section 8.

7.2 RESULTS OF THE FIRST LEVEL OF SCREENING

The results below are the conclusions reached in Section 8 of Appendix 3.

7.2.1 Iona Island

The following processes are a logical progression from the interim processes:

Option 1: Primary Treatment followed by 2 x ADWF Trickling Filter Solids Contact,

Option 2A: Primary Treatment followed by 2 x ADWF Conventional Activated Sludge,

Option 2B: Primary Treatment followed by 2 x ADWF Conventional Activated Sludge including flow from the Lions Gate Plant,

Option 3: Chemically Enhanced Primary Treatment followed by 60% of 2 x ADWF Conventional Activated Sludge.

7.2.2 Lions Gate

The following processes are a logical progression from the interim processes:

Option 1: Primary Treatment followed by 2 x ADWF Trickling Filter Solids Contact,

Option 2: Primary Treatment Followed by 2 x ADWF Biological Aerated Filter,

Option 3: 2 x ADWF High Rate Activated Sludge in parallel with Primary Treatment,

Option 4: Chemically Enhanced Primary Treatment followed by 60% of 2X ADWF Trickling Filter Solids Contact.

8 DETAILED ANALYSIS OF OPTIONS THAT PASSED FIRST LEVEL OF SCREENING

To make the results of the analysis of different processes comparable, a set of textbook process design parameters were adopted. These were applied to the analysis of the unit operations in each plant using the demand flows and loads.

Because the textbook process design parameters may differ from those experienced in the treatment plants, the analysis results may be at variance with practical experience. Once the options that passed the first round of screening have been compared and reduced to a short list, parameters experienced in the plants will be utilized for the final analysis which is reported in Appendix 10. The required upgrades and the timing and cost thereof will be reported.

The following methodology was used to analyze the build-out to secondary upgrade options:

- Preliminary process design was used for the built-out options. The worksheets for process design are included in Appendix A.
- Flows and load projections developed in Appendix 3. Upper and lower envelopes were developed and the upper envelope was used as the basis for process design. Flows and load projections included separate projections for residential, commercial and institutional, industrial, trucked liquid waste and inflow and infiltration. The
- From the results of the process design, the size and number of unit processes was estimated. This is summarized in Sections 8.1.3 for Iona Island and 8.2.3 for Lions Gate
- Based on the number and size of each unit processes, conceptual site layout were developed. See sections 8.1.4 and 8.2.4.
- For all upgrades options, estimated sludge productions were calculated as well as energy requirements.
- Preliminary capital cost estimates were then developed. Preliminary capital cost estimates for the build-out options are summarized in Section 8.1.7 for Iona Island and in Section 8.2.7 for Lions Gate. Further details on the cost estimates are included in Appendix B.
- Preliminary operating and maintenance cost estimates were developed and these incorporated sludge handling cost and energy cost. See section 8.1.8 for Iona Island and 8.2.8 for Lions Gate.

Following the second level screening, a short list of build-out to secondary options was developed. Following review comments from GVRD, the flow and load projections were modified to develop a design case that falls between the lower and upper envelope. Also the installation of additional primary clarifiers at Iona Island was deleted. Following the comments from GVRD, the above procedure was repeated for the short list of options only and the results are detailed in Appendix 10. In essence, Appendix 10 is the continuation of this Appendix 4.

8.1 IONA ISLAND

8.1.1 General

Trickling filter/solids contact (TF/SC) and conventional activated sludge (CAS) are the two processes that passed the first level of process screening and that were considered for IIWWTP secondary treatment upgrades. Chemical enhanced primary (CEP) followed by partial biological treatment, either by CAS or TF/SC, is also considered as an option to reduce the capacity of biological secondary treatment. The following four (4) upgrade options were developed with varied design capacities and secondary treatment process:

Option1: Primary + 100% of 2 x ADWF TF/SC

Option 2A: Primary + 100% of 2 x ADWF CAS

Option 2B: Primary + 100% of 2 x ADWF CAS (with diverted flow from the North Shore Sewage Area/LGWWTP)

Option 3: CEP + 60% of 2 x ADWF CAS

Further analyses of these upgrade options are detailed in this section and for the second level of process screening. Brief process descriptions, schematic flow diagrams, conceptual process designs including plant layouts, footprint requirements, sludge productions, effluent quality projections, capital and O&M cost estimates, process flexibility, and other factors are discussed in the followings sections. Following discussions at Workshop # 3, the construction of additional primary sedimentation tanks was deleted from all upgrade options.

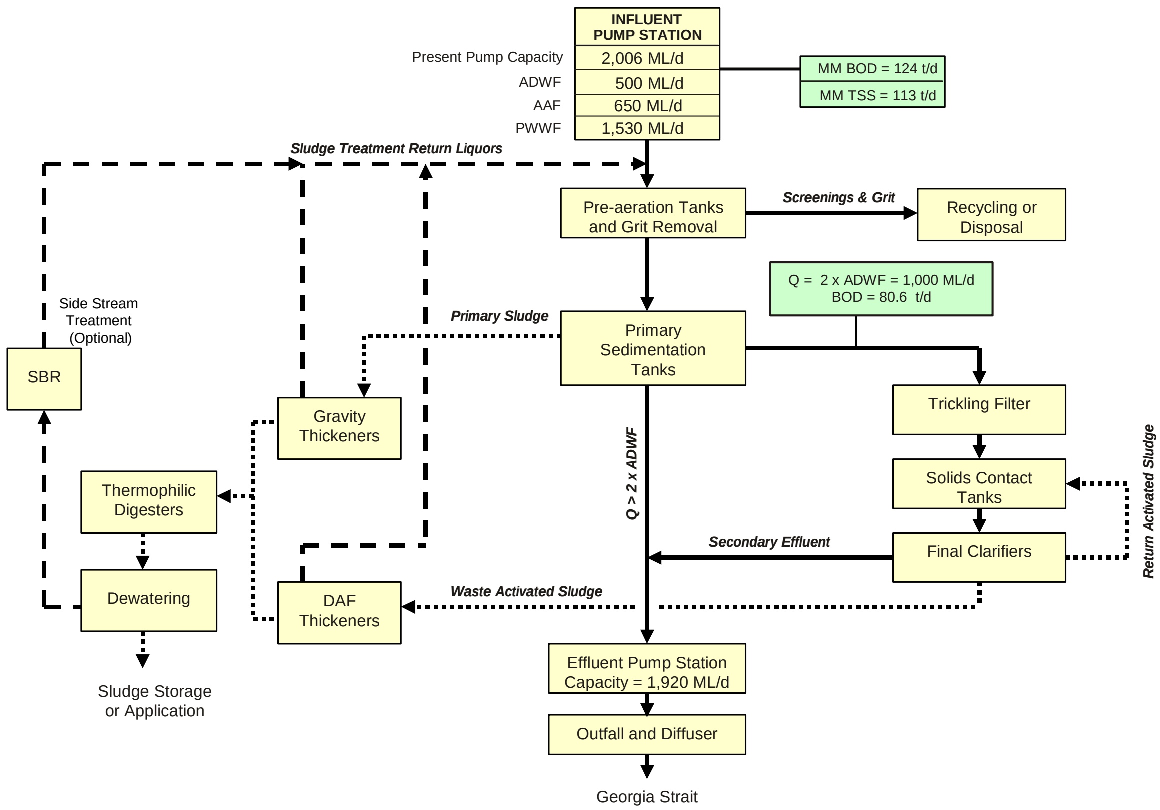
8.1.2 Description of Upgrade Options

8.1.2.1 Option 1: Primary + 100% of 2 x ADWF TF/SC

A process schematic of this option is illustrated in Figure 8.1. The preliminary (screen and grit removal) and primary (primary clarifier) treatment units are designed to treat the entire flow collected from the Vancouver Sewage Area (VSA). The TF/SC process is designed to provide two times of the average dry weather flow (ADWF) for a hydraulic capacity of 500 ML/d×2 = 1,000 ML/d, at 100% of the maximum month flow (MMF) loading of 81 t/d of BOD and 57 t/d of TSS after the primary treatment units. Final clarifiers will be used to remove TSS and the biological sludge generated from the TF/SC process. The flow in excess flow of two times the ADWF will bypass the secondary treatment units and discharge directly to the outfall pump station after the primary treatment.

The primary sludge and biological sludge are thickened in the gravity thickeners and dissolved air flotation (DAF) units, respectively. The thickened sludge from both streams will be stabilized in the same anaerobic digesters to achieve volatile solids reduction and pathogen reduction. The anaerobic digesters are designed to be operated under mesophilic or thermophilic conditions, subject to the final biosolids product and/or land application requirements, e.g. Class A or Class B biosolids. The digested biosolids will be dewatered to reduce the volume before final disposal. The rejected wastewater from the sludge handling processes (thickeners, digesters and dewatering units) is recycled back to the plant for treatment. Side-stream treatment (e.g. sequencing batch reactor process, SBR) may be an option to reduce the ammonia and organic loadings in the reject wastewater from dewatering units if required. This will be determined at the time of detailed design.

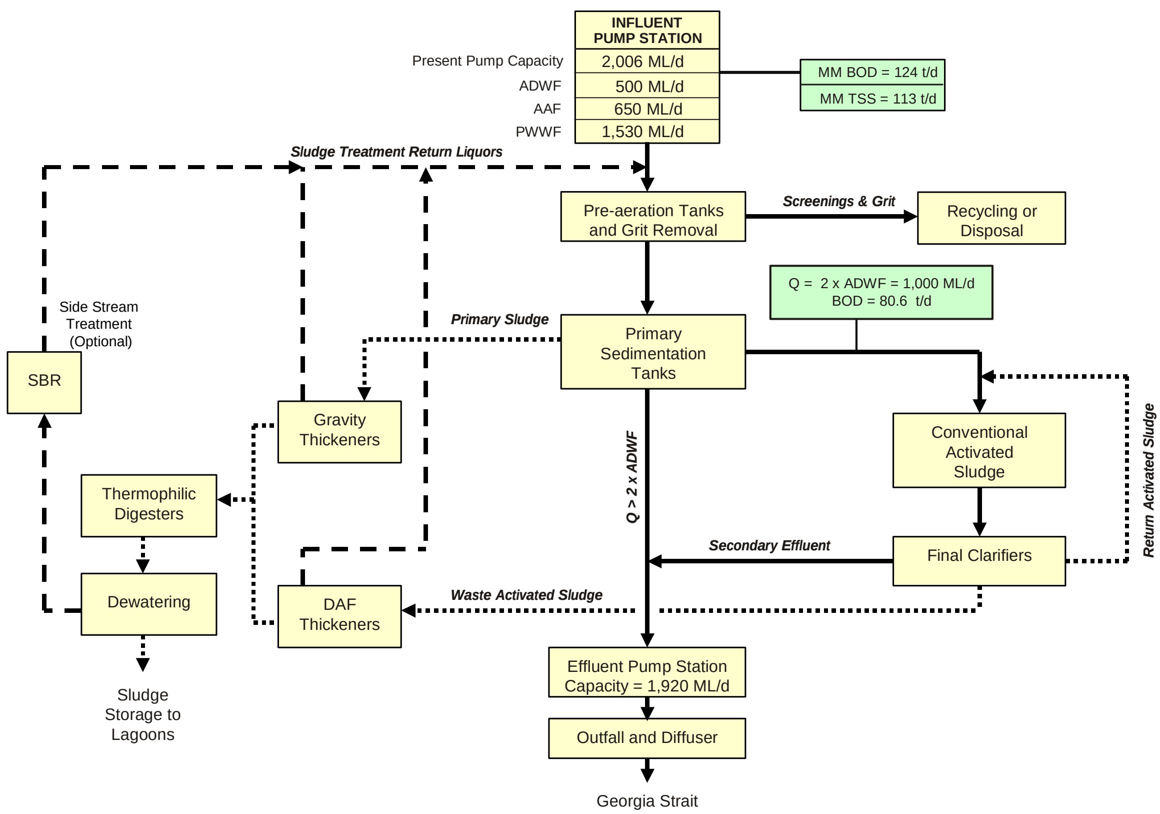
FIGURE 8.1
PROCESS SCHEMATIC OF IIWWTP UPGRADE OPTION 1



8.1.2.2 Option 2A: Primary + 100% of 2 x ADWF CAS

In this option, CAS will be used to provide secondary treatment and a process schematic is illustrated in Figure 8.2. The design flow and loads of this biological process are also two times the ADWF (500 ML/d×2=1,000 ML/d) and 100% of MMF loadings after primary treatment (81 t/d of BOD and 57 t/d of TSS). The arrangements of solids handling and reject wastewater treatment are the same as in Option 1.

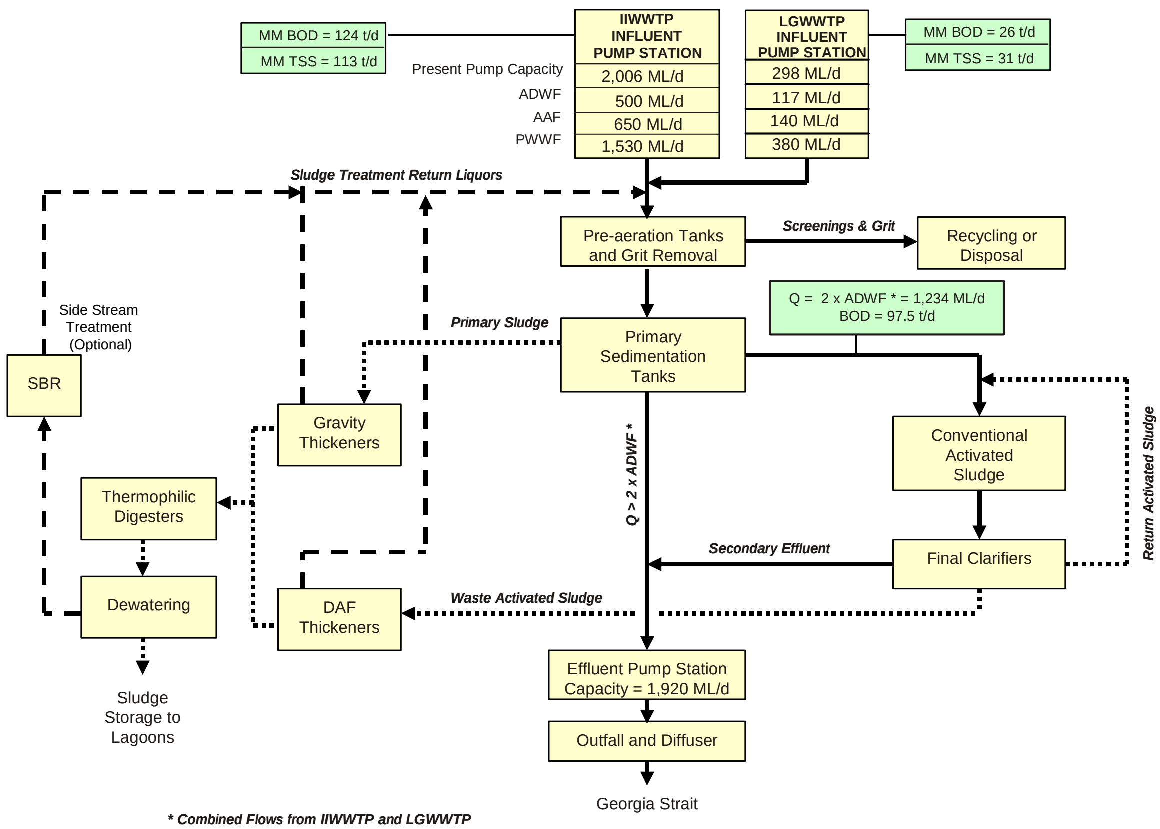
FIGURE 8.2
PROCESS SCHEMATIC OF IIWWTP UPGRADE OPTION 2A



8.1.2.3 Option 2B: Primary + 100% of 2 x ADWF CAS (with diversion from LGWWTP)

This option is the same as option 2A but with an increase in the design flow and load in order to treat both the sewage diverted from LGWWTP (NSSA) and the sewage from the VSA using CAS in the secondary treatment process. A process schematic is illustrated in Figure 8.3. Both the flows from VSA and North Shore Sewage Area (NSSA) will be treated by the preliminary and primary process, followed by the CAS process to treat two times the ADWF (1,232 ML/d) and 100% of MMF loadings after primary (98 t/d of BOD and 72 t/d of TSS). The arrangements of solids handling and reject wastewater treatment are the same as in Option 1.

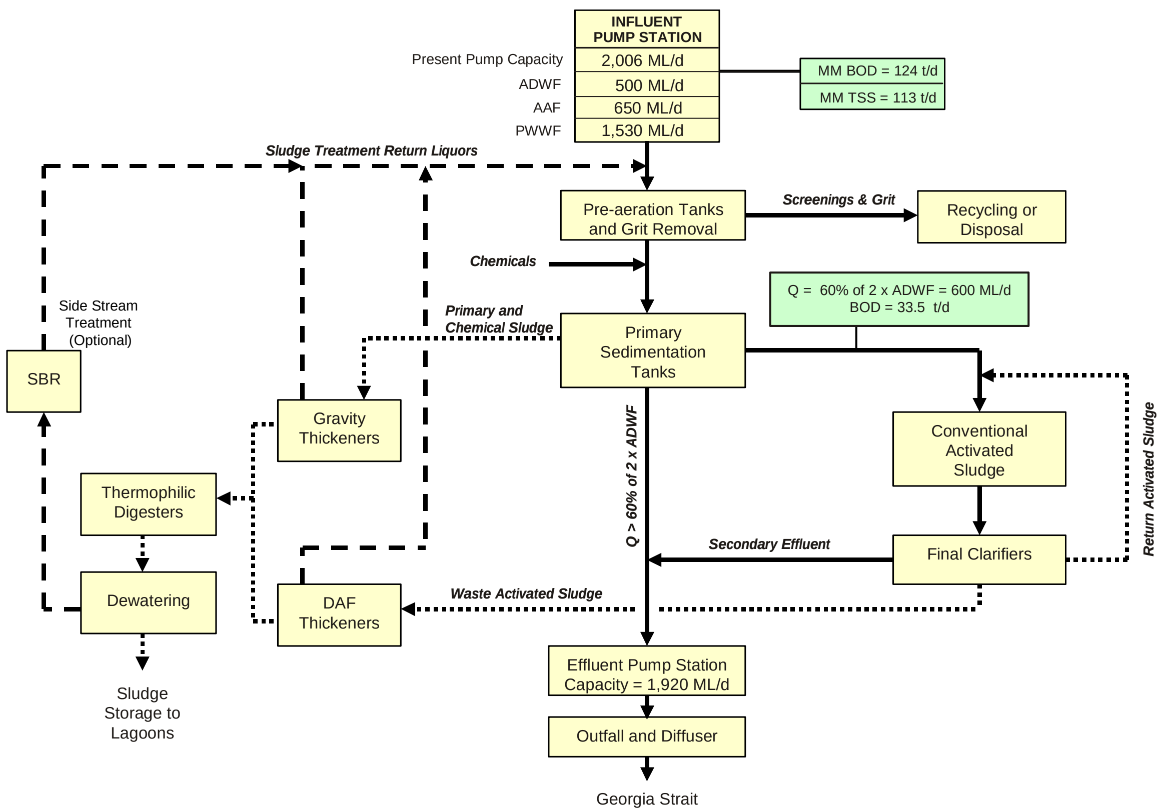
FIGURE 8.3
PROCESS SCHEMATIC OF IIWWTP UPGRADE OPTION 2B



8.1.2.4 Option 3: CEP + 60% of 2 x ADWF CAS

This option consists of using CEP to improve primary treatment efficiency thus reducing the required capacity of the secondary treatment process. A process schematic is shown in Figure 8.4. Due to higher TSS and BOD removal efficiencies in CEP (approximate 30% of TSS and 20% of BOD removal enhancements compared to conventional primary sedimentation), 60% of 2×ADWF (600 ML/d) and 60% of MMF loadings after primary (34 t/d of BOD and 14 t/d of TSS) are selected as the CAS design capacity. The primary sludge and chemical sludge will be collected in the primary sedimentation tanks and thickened in the gravity thickeners respectively.

FIGURE 8.4
PROCESS SCHEMATIC OF IIWWTP UPGRADE OPTION 3



8.1.3 Tank Size and Number of Units Required

A spreadsheet model was developed to carry out the conceptual process design and to determine the number of units required for each unit treatment process in each upgrade option. The model summary is included in Appendix A. The unit process dimensions and number of process units required are summarized in Tables 8.1 and 8.2, respectively. The actual unit dimensions are adjusted slightly in each option to obtain integral units presented in Table 8.2.

TABLE 8.1
IIWWTP UNIT PROCESS DIMENSIONS FOR EACH UPGRADE OPTION

YEAR	2036 Build-out to Secondary			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Primary Clarifier				
Length (m)	66.0	66.0	66.0	66.0
Width (m)	15.0	15.0	15.0	15.0
Depth(m)	2.7	2.7	2.7	2.7
Unit Size (m^2)	990.0	990.0	990.0	990.0
Existing Surface area (m^2)	11819	11819	11819	11819
Total Area Required (m^2)	16250	16250	19725	16250
Aeration/Solids Contact Tank				
Length (m)	86.0	86.0	86.0	86.0
Width (m)	30.0	30.0	30.0	30.0
Depth (m)	5.5	5.5	5.5	5.5
Unit Size (m^2)	2580	2580	2580	2580
Total Area Required (m^2)	6019	40300	48750	19929
Trickling Filter (TF)				
Diameter (m)	44.0			
Depth (m)	6.0			
Unit Size (m^2)	1520			
Total Volume Required (m^3)	8396			
Final Clarifier				
Diameter (m)	44.0	44.0	44.0	44.0
Depth (m)	4.5	4.5	4.5	4.5
Unit Size (m^2)	1520	1520	1520	1520
Total Area Required (m^2)	23333	23333	28747	14000
Gravity Thickener				
Diameter (m)	19.8	19.8	19.8	19.8
Depth (m)	3.0	3.0	3.0	3.0
Unit Size (m^2)	308	308	308	308
Existing Surface area (m^2)	616	616	616	616
Total Area Required (m^2)	565	565	720	1056
DAF Thickener				
Diameter (m)	20.0	20.0	20.0	20.0
Depth (m)	3.5	3.5	3.5	3.5
Unit Size (m^2)	314	314	314	314
Total Area Required (m^2)	1000	986	1192	299
Digester				
Diamater (m)	32.0	32.0	32.0	32.0
Depth (m)	10.6	10.6	10.6	10.6
Unit Size (m^3)	8521	8521	8521	8521
Existing Volume (m^3)	19816	19816	19816	19816
Total Volume Required (m^3)	37520	37230	46116	37814
Centrifuge				
Unit capacity (m^3/h)	72.0	72.0	72.0	72.0
Digested sludge volume (m^3/d)	2501	2482	3074	2521

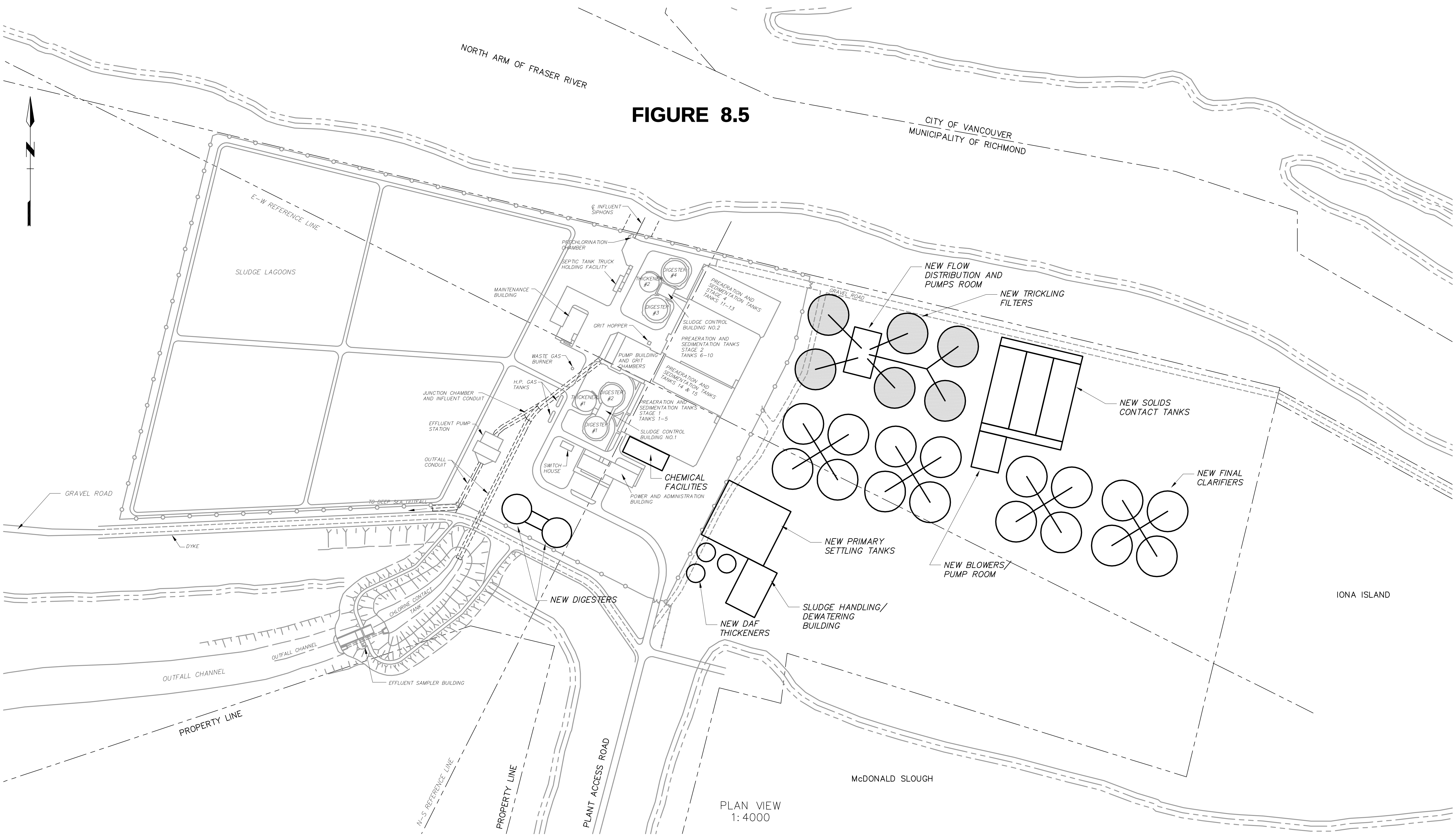
TABLE 8.2
IIWWTP NUMBER OF UNITS REQUIRED FOR EACH UPGRADE OPTION

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Primary Clarifier				
Total Requirement	20	20	23	20
Existing	15	15	15	15
Addition	5	5	8	5
Aeration/Solids Contact Tank				
Total Requirement	3	16	19	8
Existing	0	0	0	0
Addition	3	16	19	8
Trickling Filter				
Total Requirement	6			
Existing	0			
Addition	6			
Final Clarifier				
Total Requirement	16	16	19	10
Existing	0	0	0	0
Addition	16	16	19	10
Gravity Thickener				
Total Requirement	2	2	3	4
Existing	2	2	2	2
Addition	0	0	1	2
DAF Thickener				
Total Requirement	3	3	4	1
Existing	0	0	0	0
Addition	3	3	4	1
Digester				
Total Requirement	6	6	7	6
Existing	4	4	4	4
Addition	2	2	3	2
Centrifuge				
Total Requirement	7	7	9	8
Existing	0	0	0	0
Addition	7	7	9	8
BNR (optional)				
Total Requirement	2	1	2	1
Existing	0	0	0	0
Addition	2	1	2	1

8.1.4 Conceptual Site Layout

Conceptual site layouts for each upgrade option are illustrated in Figures 8.5, 8.6, 8.7 and 8.8. A modular concept is proposed for the ease of expanding from the interim treatment to build-out to secondary. For example in the CAS option, three additional modules of bioreactors and final clarifiers are proposed to expand the biological treatment capacity from interim (50% of ADWF) to build-out to secondary (100% of 2×ADWF).

FIGURE 8.5



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December 03, 2003 2:07 p.m.

Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103 | ACAD DWG. SECONDARY-1 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

BUILDOUT TO SECONDARY

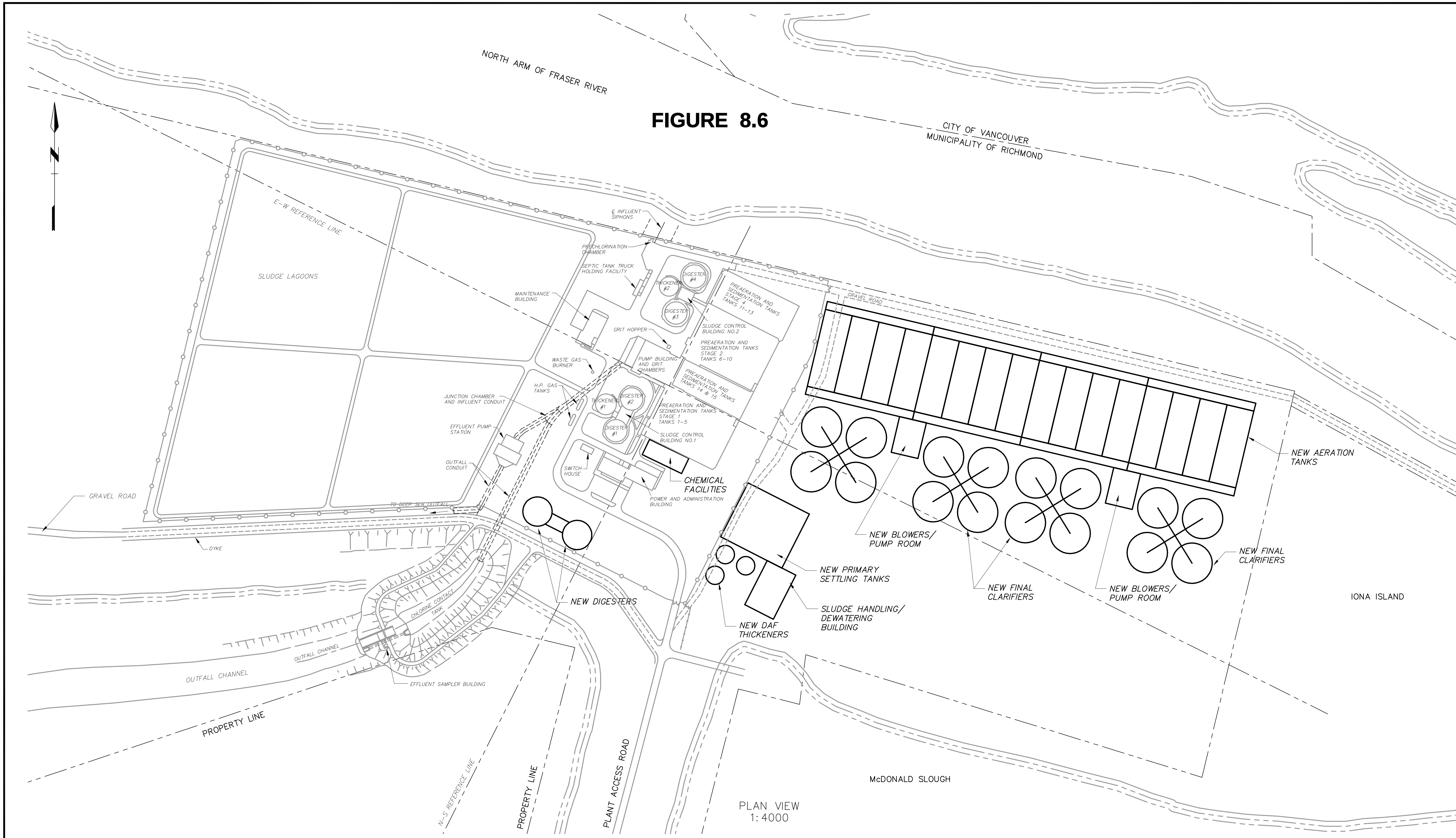
OPTION 1
PRIMARY + 100% OF 2 x ADWF TF/SC

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:
SAROS DWG. ID No.:

FIGURE 8.6



Dayton & Knight Ltd.

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DWG. No. 415-1-103 ACAD DWG. SECONDARY-2A 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

BUILDOUT SECONDARY

OPTION 2A
PRIMARY + 100% OF 2 x ADWF CAS

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

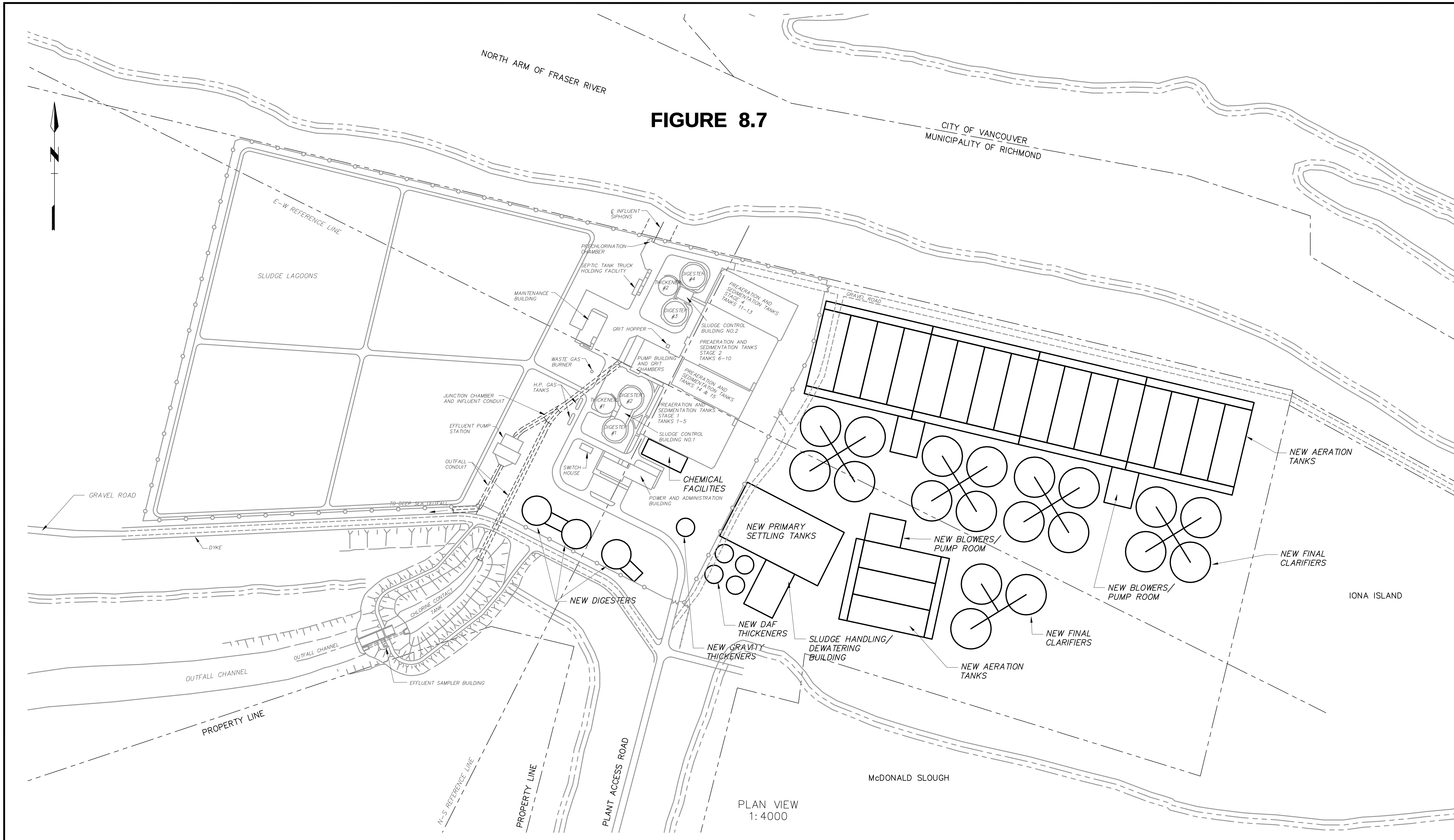
SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:

Xrefs: Option.DWG;
Drawing: V:\117\ACTIVE\11700018\TASK 3-4 PROCESSES\WWTP LAYOUT\SECONDARY-2A.DWG
December 03, 2003 2:09 p.m.

FIGURE 8.7



Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103 ACAD DWG. SECONDARY-2B 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chk'd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

BUILDOUT SECONDARY

OPTION 2B
PRIMARY + 100% OF 2 x ADWF CAS
(+ LGWWTP)

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

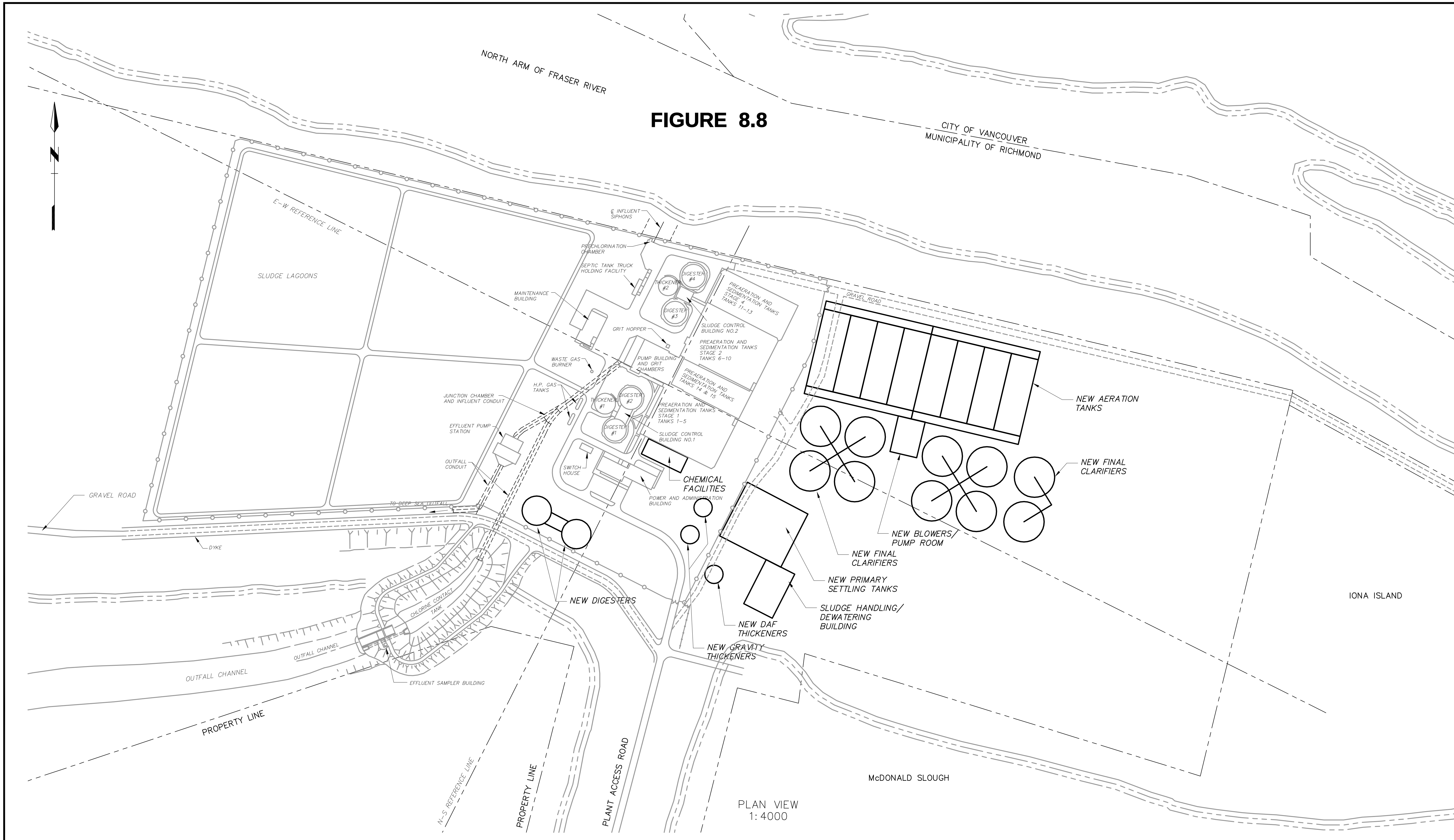
SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:

Xrefs: Option.DWG;
Drawing: V:\117\ACTIVE\11700018\TASK 3-4 PROCESSES\WWTP LAYOUT\SECONDARY-2B.DWG
December 03, 2003 2:10 p.m.

FIGURE 8.8



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Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-103 ACAD DWG. SECONDARY-3 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: JC
Drawn: AA
Checked:
Approved
Manager

IONA ISLAND WWTP
UPGRADING PLAN

BUILDOUT TO SECONDARY

OPTION 3
CEP + 60% OF 2 x ADWF CAS

SCALE: AS SHOWN
DISTRICT FILE
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:
SAROS DWG. ID No.:

New primary and secondary clarifiers, DAF, and dewatering units are located on land owned by GVS&DD east of the existing plant. The initial proposal was that the new digesters and gravity thickeners would be located south and southeast of the existing plant, respectively. However, it was later determined that the use of the northeast lagoon would be used for facility expansion. An addition to the administration building and control room can be built by extending the southeast wing of existing building. Chemical storage and associated facilities can be constructed at the existing location subject to requirements for pre-loading in case of high tanks.

For comparison purposes, the approximate total footprint requirements of each upgrade option are shown in Table 8.3 (actual reactors/building footprint plus 20% of spacing, roads and miscellaneous). With a design capacity of 1,000 ML/d, the CAS options (Option 2A) requires 15% more footprint than the TF/SC option (Option 1). With the flow diverted from NSSA in Option 2B, the footprint requirement increases by 21% using the CAS process. Option 3 requires 38% less footprint than Option 2A, and 10% less footprint than Option 1 since only 60% of 2 x ADWF would receive biological treatment.

TABLE 8.3
IIWWTP FOOTPRINT REQUIREMENTS FOR EACH UPGRADE OPTION

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Total Footprint Required (m^2)	63,400	92,700	112,300	57,700

8.1.5 Projected Effluent Quality

The projected effluent BOD and TSS concentrations of these four options are summarized in Table 8.4 for the design flow (2×ADWF) and PWWF conditions. At the design flow condition (2×ADWF), the effluent BOD and TSS concentrations are about 20 mg/L, respectively. The estimated PWWF effluent concentrations will be higher than the design flow condition due to the load re-distributions between the biological treatment portion and primary treatment portion. Better effluent quality can be expected at the design AAF condition. Substantial effluent toxicity reduction can be expected for all treatment options as a result of the provision of biological treatment.

TABLE 8.4
IIWWTP EFFLUENT CONCENTRATION PROJECTIONS FOR EACH UPGRADE OPTION

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Design PWWF				
Effluent BOD (mg/L)	31	31	31	30
Effluent SS (mg/L)	26	26	26	17
Design Flow (2xADWF)				
Effluent BOD (mg/L)	20	20	20	20
Effluent SS (mg/L)	20	20	20	20

8.1.6 Sludge Production Projection

The projected sludge production for each upgrade option is shown in Table 8.5 for the primary, chemical and biological sludge respectively. The raw sludge quantities are expressed in dry solids. The estimated sludge volumes (m^3/d) at varied sludge handling stages are also shown in Table 8.5, including the gravity thickener underflow, DAF supernatant, digested sludge and dewatered sludge. The increase in sludge production compared to current averages of 970 m^3/d of raw digested sludge (estimated maximum month) and 84 m^3/d dewatered sludge at 35% solids concentration, are also summarized in Table 8.6, for their dry weight and bulk volumes, respectively.

Option 3 will produce the largest volumes of sludge as a consequence of the CEP process (except Option 2B will treat the sewage from both the VSA and NSSA). Option 1 and Option 2A are expected to produce equivalent amounts of sludge. Option 2B will produce more sludge compared with Option 1 and 2A, because of the increased flow and loads diverted from NSSA. With the provision that existing sludge stockpiling space will be used for plant expansion and existing lagoon capacity will not be sufficient to provide the dewatering, mechanical or other types of dewatering will be required to dewater the digested sludge prior to hauling. The digested sludge and dewatered sludge concentrations are estimated about 3% and 27%, by using thermophilic anaerobic digestion and centrifuge respectively.

TABLE 8.5
IIWWTP SLUDGE PRODUCTION FOR EACH UPGRADE OPTION
(MAXIMUM MONTH LOAD)

YEAR		**2036 Build-out to Secondary**			
Option	Unit	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Raw Sludge/Biosolids					
Primary Sludge	T/d	57	57	72	-
CEP Sludge	T/d	0	0	0	106
Secondary Biosolids	T/d	48	47	57	14
Total Raw Sludge	T/d	104	104	129	120
Thickened Sludge					
Gravity Thickener	m^3/d	1130	1130	1440	2111
DAF Supernatant	m^3/d	1371	1352	1634	410
Total Thickened Sludge	m^3/d	2501	2482	3074	2521
Digested Sludge	m^3/d	2501	2482	3074	2521
Dewatered Sludge	m^3/d	266	265	329	306

TABLE 8.6
IIWWTP INCREASES OF SLUDGE PRODUCTION COMPARED TO CURRENT LEVEL
(MAXIMUM MONTH LOADS)

YEAR		**2036 Build-out to Secondary**			
Option	Unit	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Raw Sludge (by weight)	%	115	114	166	147
Thickened Sludge	%	158	156	217	160
Digested Sludge	%	158	156	217	160
Dewatered Sludge	%	217	215	292	263

8.1.7 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 8.7. Detailed breakdowns of the process design are included in Appendix B. Option 3 with CEP and partial CAS treatment apparently requires the lowest capital expenditure. For the options with partial biological treatment only, the TF/SC option has the lowest capital cost, followed by the CAS option. Option 2B to treat both VSA and NSSA sewage require the highest capital cost, but this cost should be considered in accordance with the flow diversion decision and other engineering/social factors.

If the interim upgrade Option 2 (50% ADWF TF) or interim Option 4 (CEP + 50% ADWF RTF) were to be selected, the treatment equipment built in the interim upgrades would be incorporated in the built-out Option 1 (TF for 2 x ADWF) and the estimated cost of build-out Option 1 would be lower. Similarly, if interim upgrade Option 1 (50% ADWF CAS) or interim Option 3 (50% ADWF HRAS), the treatment equipment would be incorporated into built-out Option 2 (CAS) or Option 3 (CEP + 60% of 2 x ADWF CAS) and the cost of the built-out options would be reduced. However, no interim option has been selected at this time. For this reason, the cost estimates for build-out options were developed as stand alone estimates.

TABLE 8.7
IIWWTP CAPITAL COSTS OF EACH UPGRADE OPTION

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
CAPITAL COSTS				
Site Improvements	$43,814,000	$46,242,000	$53,061,000	$36,370,000
Primary Clarifiers	$0	$0	$0	$0
Chemical Feed	$0	$0	$0	$1,500,000
Aeration Basin/Solids Contact	$15,325,200	$81,734,400	$97,059,600	$40,867,200
Trickling Filter	$50,358,960	$0	$0	$0
Secondary Clarifiers	$52,044,800	$52,044,800	$61,803,200	$32,528,000
Gravity Thickeners	$0	$0	$1,386,000	$2,772,000
DAF Thickeners	$21,301,760	$20,636,080	$25,295,840	$6,656,800
Digesters	$16,818,480	$16,017,600	$24,827,280	$16,818,480
Mechanical Dewatering	$9,960,000	$9,960,000	$12,800,000	$11,300,000
Site Works	$12,010,500	$11,170,500	$12,800,000	$8,429,400
Admin/Maint. Building	$8,000,000	$8,000,000	$8,000,000	$8,000,000
Control System	$6,233,968	$7,215,715	$8,926,877	$4,437,699
Electrical substation	$1,500,000	$1,500,000	$1,500,000	$1,500,000
Odour Control	$1,000,000	$0	$0	$0
Existing Facility Upgrades	$0	$0	$0	$0
Division 1 Cost	$4,863,842	$5,206,977	$6,359,970	$3,370,239
Engineering	$38,138,827	$40,723,375	$49,193,567	$27,388,733
Project Management/QA/QC	$9,534,707	$10,180,844	$12,298,392	$6,847,183
Contingency	$71,510,300	$76,356,329	$92,237,939	$51,353,874
Sub-Total	$362,415,344	$386,988,620	$467,549,665	$260,139,608
Net GST, 0% of Sub-Total	$0	$0	$0	$0
Total Capital Costs	$362,416,000	$386,989,000	$467,550,000	$260,140,000

8.1.8 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs for each upgrade option are shown in Table 8.8. The existing primary plant has a staff of 57 persons. For options 1, 2A and 3 it is estimated that the staff would increase to 80 full-time persons. The chemical costs in Option 1, 2A and 2B are for the polymer used for dewatering uses (i.e. centrifuge). The chemicals in Option 3 include the alum and polymer used for CEP as well as the polymer needed for dewatering. No chemical disinfection (e.g. chlorination) is considered in this upgrade. The residual management costs are estimated based on a rate of $100/tonne for hauling, recycling (e.g. land application), and other fixed expenses, if application sites are available. The solids concentration is estimated about 30% using mechanical dewatering by centrifuge.

TABLE 8.8
IIWWTP OPERATING AND MAINTENANCE COSTS OF EACH UPGRADE OPTION

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
O&M COSTS				
Labour	$5,365,000	$5,365,000	$5,365,000	$5,365,000
Chemical Costs	$525,000	$521,000	$649,000	$7,524,000
Residuals Management	$8,099,000	$8,047,000	$10,014,000	$9,293,000
Energy/Power	$1,659,000	$3,332,000	$3,847,000	$2,343,000
Repair/Maintenance	$5,675,000	$5,872,000	$6,516,000	$4,857,000
Administration and others	$1,274,000	$1,274,000	$1,358,000	$1,303,000
Total (O&M Costs)	$22,597,000	$24,412,000	$27,750,000	$30,686,000

8.1.9 Life Cycle Cost Estimates

The life cycle costs (LCC) of each upgrade option are estimated in Table 8.9, using a 6% real discount rate and 30 years as the planning period. By assuming that the construction of the build-out to secondary will commence in 2018, Option 2B has the highest LCC at present worth, followed by Option 3, 2A, and Option 1. It should be noted that Option 2B provides additional treatment capacity for NSSA diversion. Because of CEP and partial biological treatment, Option 3 has higher LCC than Option 1 and 2A with biological treatment only.

TABLE 8.9
IIWWTP LIFE CYCLE COST ESTIMATE FOR EACH UPGRADE OPTION

YEAR	**2036 Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF
Total 30-yr. O&M Costs	$126,264,000	$136,406,000	$155,058,000	$171,463,000
Discounted Capital Costs	$142,825,000	$152,509,000	$184,258,000	$102,519,000
Total Capital and O&M Costs at present value	$269,090,000	$288,916,000	$339,316,000	$273,982,000

8.1.10 Flexibility of Phasing

The plant development can be facilitated by planning modular expansions from interim to build-out to secondary. With the CAS option, the unit processes can be expanded from one module (250 ML/d) during the interim treatment to four modules (1,000 ML/d) in the build-out to secondary stage. By extending the height of roughing trickling filter (RTF) units and adding extra units of TF/SC units, the RTF option can be expanded during the interim to meet the design capacity of build-out to secondary using TF/SC process. Additional sludge handling units can be phased in when needed. All capital investment during the interim stage can be used in the build-out to secondary stage.

8.1.11 Energy Requirement

The major energy requirements for operating the secondary process are the various pumps and blowers for aeration. The existing influent and effluent pumps are still necessary to introduce and discharge the wastewater. Additional pumps are needed to increase the hydraulic gradient after the primary process (primary effluent) and ensure gravity flow through the bioreactors and final clarifiers to the effluent pump chamber. Additional pumps are needed in the TF/SC option to introduce the primary effluent to the top of TF tower. Additional pumps are required for internal recycling (e.g. return activated sludge) and sludge handling operation (e.g. wasted activated sludge, scum collection, DAF, anaerobic digesters, and dewatering).

The energy requirements of each upgrade option are estimated in Table 8.10. The electricity costs are based on pumps, blowers, and mechanical operation, at 2003 BC Hydro Business Rate structure. The natural gas expenditure for digester heating is estimated about 10% of total electricity bill (2002 record) at a rate of $11/GJ. Digester exhaust (50 to 60% methane under normal condition) can be used for the boilers and co-generation engine operation to recovery energy.

TABLE 8.10
IIWWTP ENERGY REQUIREMENT OF EACH UPGRADE OPTION

YEAR	**2036 Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Energy Requirement				
Electricity, kWh/yr	27,644,000	55,539,000	64,113,000	35,496,000
Natural Gas, GJ/yr	251,300	504,900	582,800	322,700
Electricity Cost	$1,508,000	$3,029,000	$3,497,000	$2,130,000
Natural Gas Cost	$151,000	$303,000	$350,000	$213,000
Total Energy Cost	$1,659,000	$3,332,000	$3,847,000	$2,343,000

8.1.12 Ability to Handle Load Variability

Both CAS and TF/SC have demonstrated an excellent capability to handle loading variability within a reasonable range. HRAS has less tolerance to shock loading than the other two processes. This is the result of the shorter hydraulic retention time and sludge age.

CAS can be further upgraded to biological nutrient removal (BNR) to remove nitrogen and phosphorus with the addition of compartments in the bioreactor and internal recycle arrangement. TF/SC can be modified to achieve nitrification with internal flow recirculation, but has limited capacity for denitrification and phosphorus removal. HRAS process can be retrofitted for BNR but more infrastructure expansion is to be expected (e.g. more tankage, aeration and pumping capacity).

8.1.13 Visual Impact

The CAS and HRAS options will not cause any more adverse virtual impact than the existing primary plant. The TF/SC option with its 6 metre high tanks will result in similar virtual impact as the Annacis Island and Lulu island WWTPs. Green belt setback (vegetation or fence) can be considered to mitigate any visual impact on the adjacent public recreational area.

8.2 LIONS GATE

8.2.1 General

Trickling filter/solids contact (TF/SC), biological aerated filter (BAF) and high rate activated sludge (HRAS) are the three processes which passed the first level of process screening and are considered for the LGWWTP secondary treatment upgrade. Chemically enhanced primary (CEP) followed by partial biological treatment, is also considered a potential scheme to reduce the required capacity of biological secondary treatment. For the purposes of this report TF/SC has been evaluated as the process following CEP. During the investigation it was determined that HRAS required more space than was available on the site. A smaller footprint process was therefore required. The process chosen for investigation is the TF/SC in parallel with Primary treatment. The following five (5) upgrade options were developed with secondary design capacities in accordance with the MSR (2 x ADWF) and one process utilizing CEP and a lower biological capacity .

- Option1: Primary + 100% of 2 x ADWF TF/SC,
- Option 2: Primary + 100% of 2 x ADWF BAF,
- Option 3: 2 x ADWF HRAS + Primary,
- Option 4: CEP + 60% of 2 x ADWF TF/SC,
- Option 5: 100% of 2 x ADWF TF/SC in parallel with Primary.

Further analysis of these upgrade options is detailed in this section. Brief process descriptions, schematic flow diagrams, conceptual process designs and plant layouts, footprint requirements, sludge production, effluent quality projections, capital and O&M cost estimates are presented. Process flexibility, environmental and social impacts and other factors are discussed in the following sections.

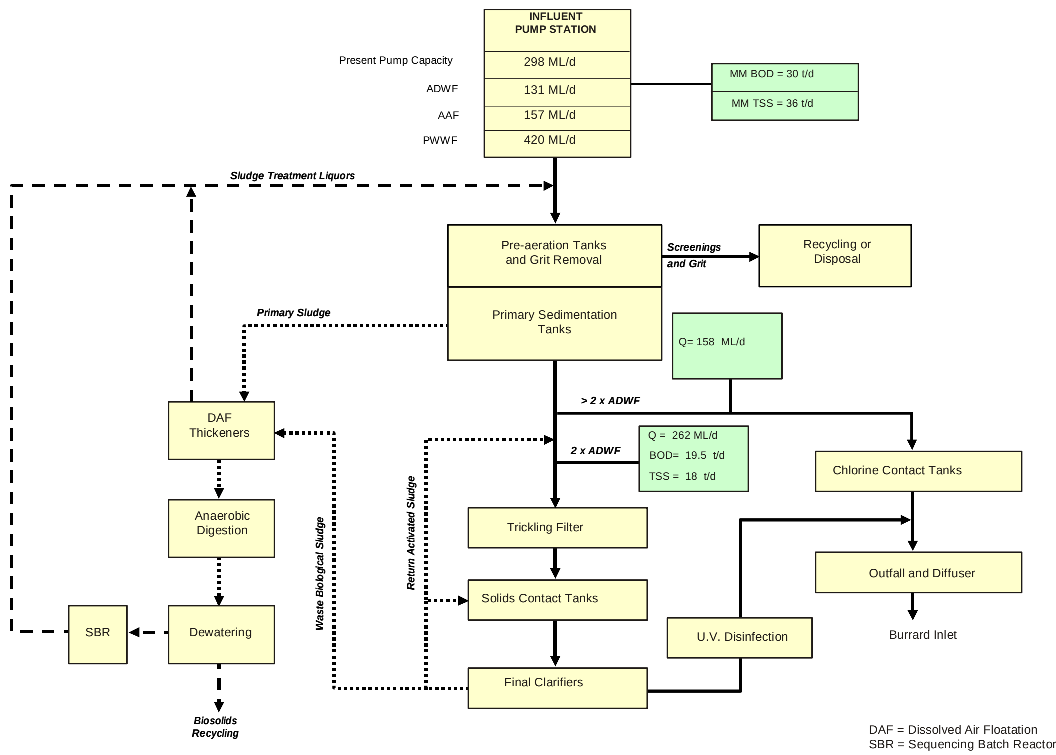
8.2.2 Description of Upgrade Options

8.2.2.1 Option 1: Primary + 100% of 2 x ADWF TF/SC

A process schematic of this option is illustrated in Figure 8.9. The preliminary (screen and grit removal) and primary (primary clarifier) units are designed to treat the entire flow collected from the North Shore Sewage Area (NSSA). The TF/SC process is designed to provide two times the average dry weather flow (ADWF) capacity (131 ML/d × 2 = 262 ML/d), at 100% of the maximum month flow (MMF) loading (19.5 t/d of BOD and 18 t/d of TSS) after the primary treatment units. Final clarifiers will be used to remove TSS and biosolids (biological sludge) generated from the TF/SC process. Secondary treated effluent will receive UV irradiation for pathogen reduction. Flows greater than two times the ADWF will, after the primary treatment, bypass the secondary treatment units and be discharged directly to the chlorination system and outfall.

The primary sludge and biological sludge are co-thickened in the dissolved air flotation (DAF) thickeners. The thickened sludge will be stabilized, as it is at present, in anaerobic digesters to achieve volatile solids and pathogen reduction. The anaerobic digesters are designed to be operated at mesophilic or thermophilic condition, subject to the final biosolids product and/or land application requirements, e.g. Class A or Class B biosolids. The digested biosolids will be dewatered, as at present, to reduce the volume before transporting to beneficial reuse. The effluents from the sludge handling processes (thickeners, digesters and dewatering units) are recycled to the plant for treatment. The centrate from dewatering of the digested sludge could be treated in a sequencing batch reactor (SBR) only if it is necessary to reduce the ammonia before discharging the effluent into the receiving environment.

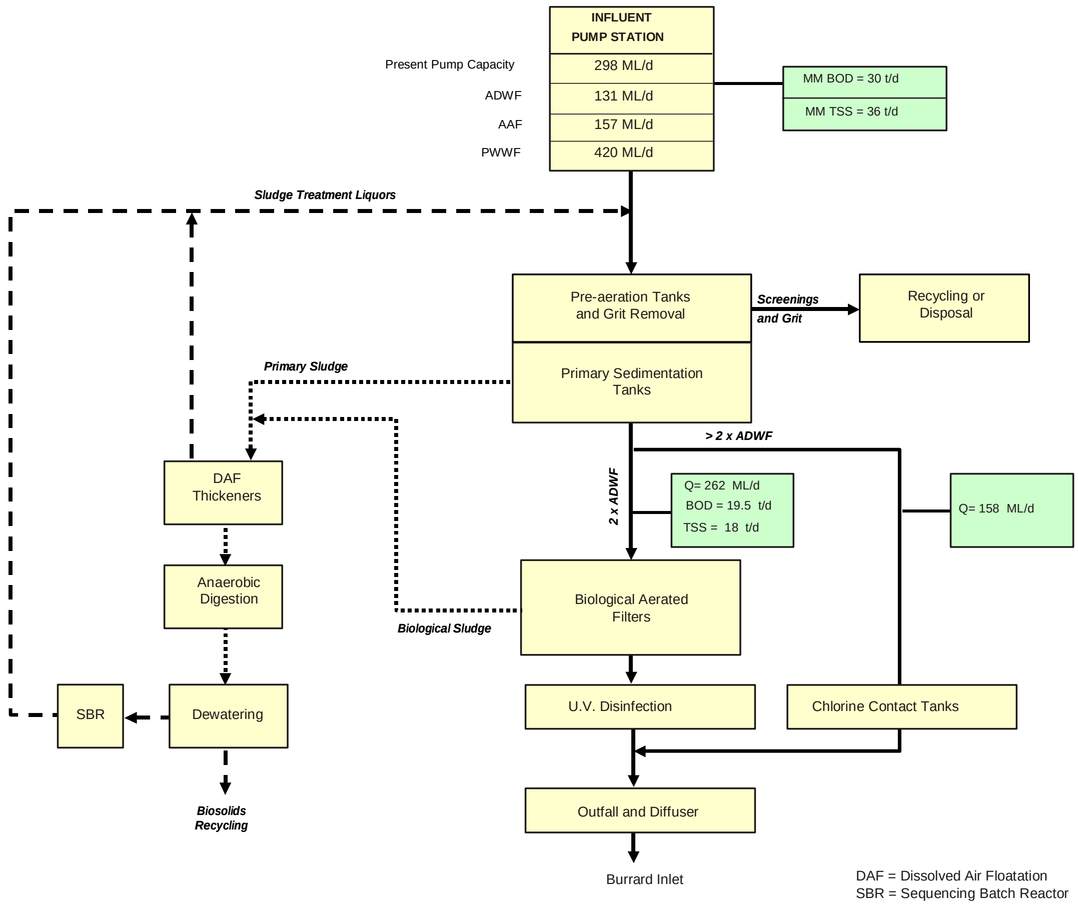
FIGURE 8.9
PROCESS SCHEMATIC OF LGWWTP UPGRADE OPTION 1



8.2.2.2 Option 2: Primary + 100% of 2 x ADWF BAF

In this option, BAF will be used to provide secondary treatment. A process schematic is illustrated in Figure 8.10. The design flow and loads for this biological process are two times the ADWF (131 ML/d × 2 = 262 ML/d) and 100% of MMF loadings after primary (19.5 t/d of BOD and 18 t/d of TSS). The BAF system does not require final clarifiers and the flows from the secondary and primary treatment systems will be handled in the same way as in Option 1. The arrangements for solids handling and sludge treatment effluent are the same as in Option 1.

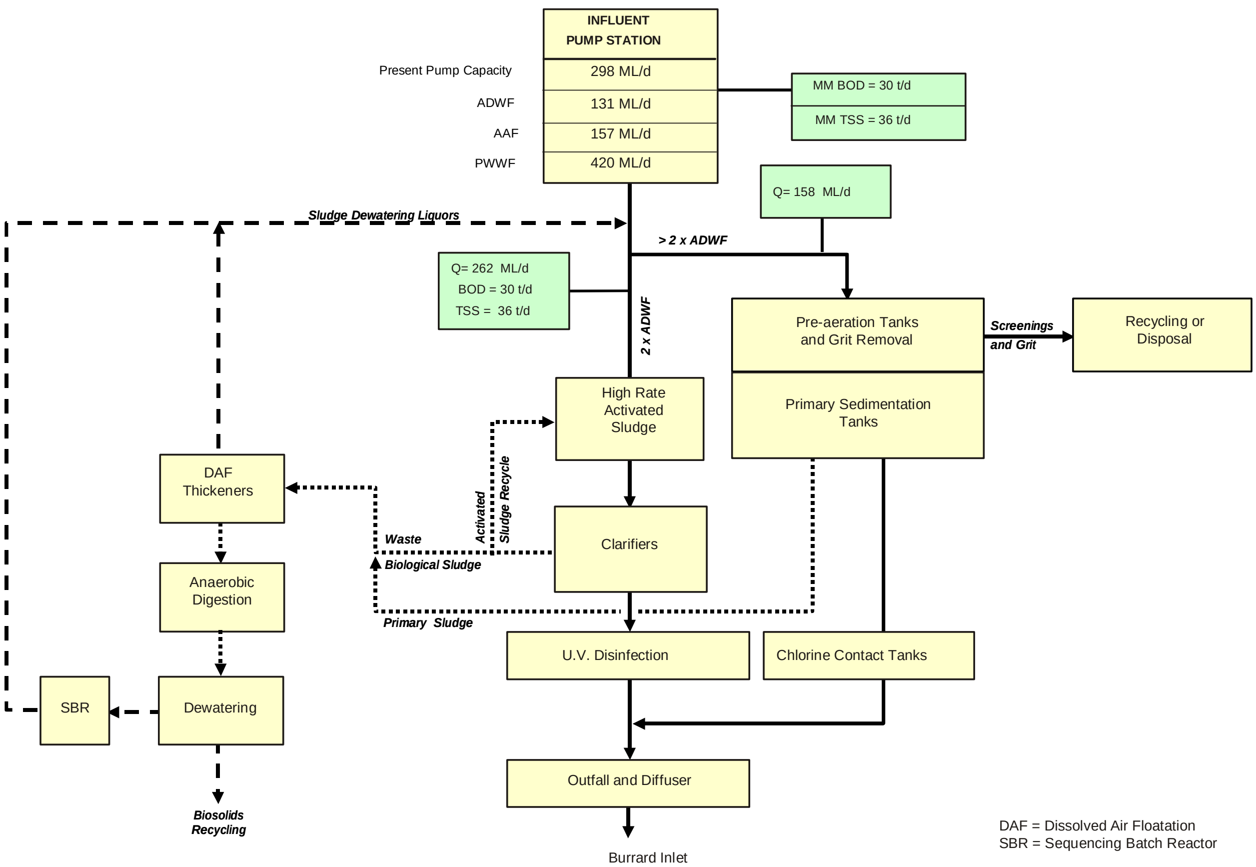
FIGURE 8.10
PROCESS SCHEMATIC OF LGWWTP UPGRADE OPTION 2



8.2.2.3 Option 3: Primary + 100% of 2 x ADWF HRAS

HRAS will be used in conjunction with primary treatment in this option. Figure 8.11 shows a process schematic with the HRAS and primary treatment processes in parallel. Following preliminary screening, a flow of 200% of ADWF is passed through a grit removal facility. A flow of 200% of ADWF (200% x 131 ML/d = 262 ML/d) will be directed to the HRAS process for treatment. The remaining flow (greater than 262 ML/d) will be treated in the primary units only. Flows from the secondary and primary treatment systems will be handled in the same way as in Option 1. The arrangements for solids handling and sludge treatment effluent are the same as in Option 1.

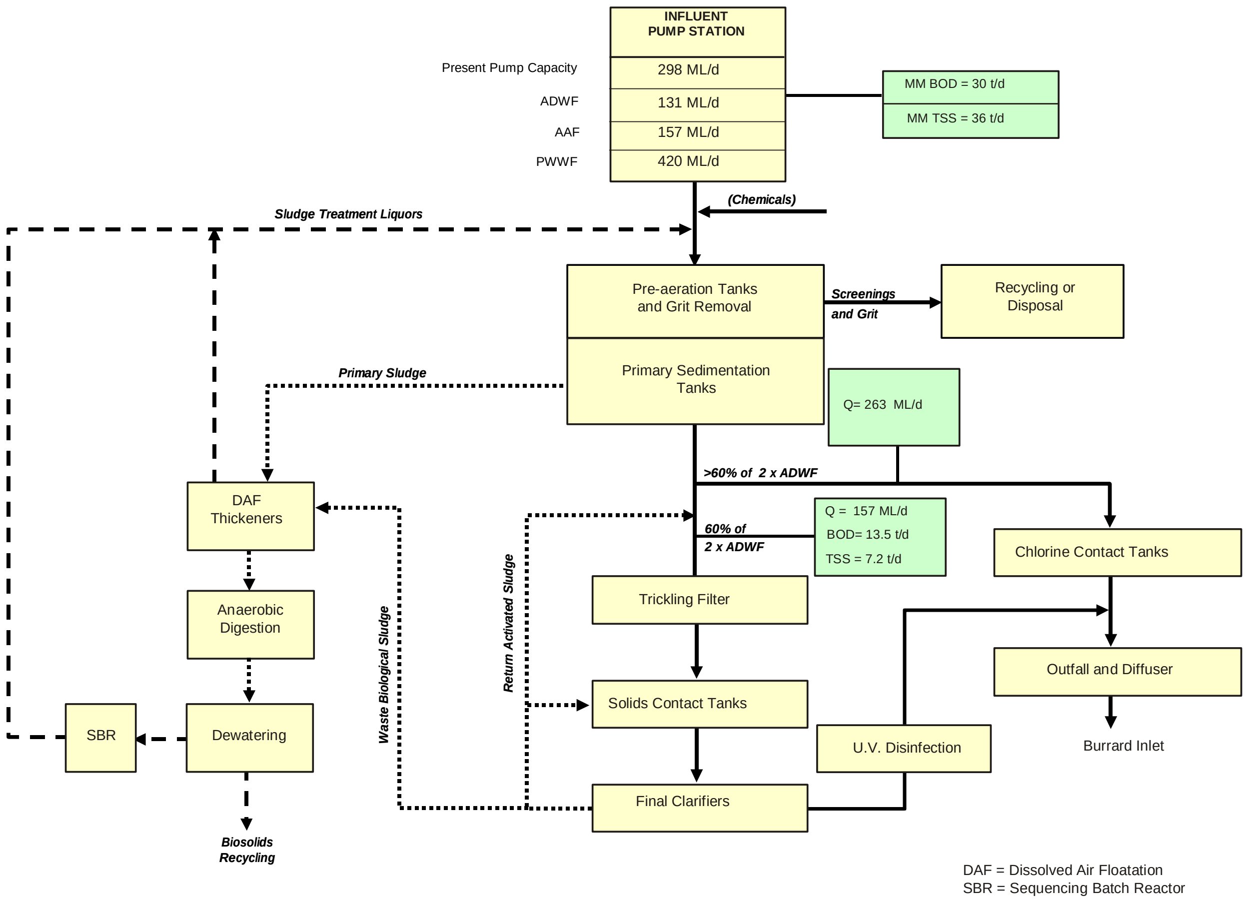
FIGURE 8.11
PROCESS SCHEMATIC OF LGWWTP UPGRADE OPTION 3



8.2.2.4 Option 4: CEP + 60% of 2 x ADWF TF/SC

CEP will be used in this option to improve the primary treatment efficiencies and reduce the secondary treatment capacity required. A process schematic is shown in Figure 8.12. Due to higher TSS and BOD removal efficiencies in CEP, 60% of 2×ADWF (157 ML/d) and 60% of MMF loadings after primary (13.5 t/d of BOD and 7.2 t/d of TSS) are selected as the TF/SC design capacity. The primary sludge and chemical sludge will be collected in the primary sedimentation tanks and co-thickened with the secondary sludge in the DAF thickeners. Flows from the secondary and primary treatment systems will be handled in the same way as in Option1. The arrangements for solids handling and sludge treatment effluent are the same as in Option 1.

FIGURE 8.12
PROCESS SCHEMATIC OF LGWWTP UPGRADE OPTION 4

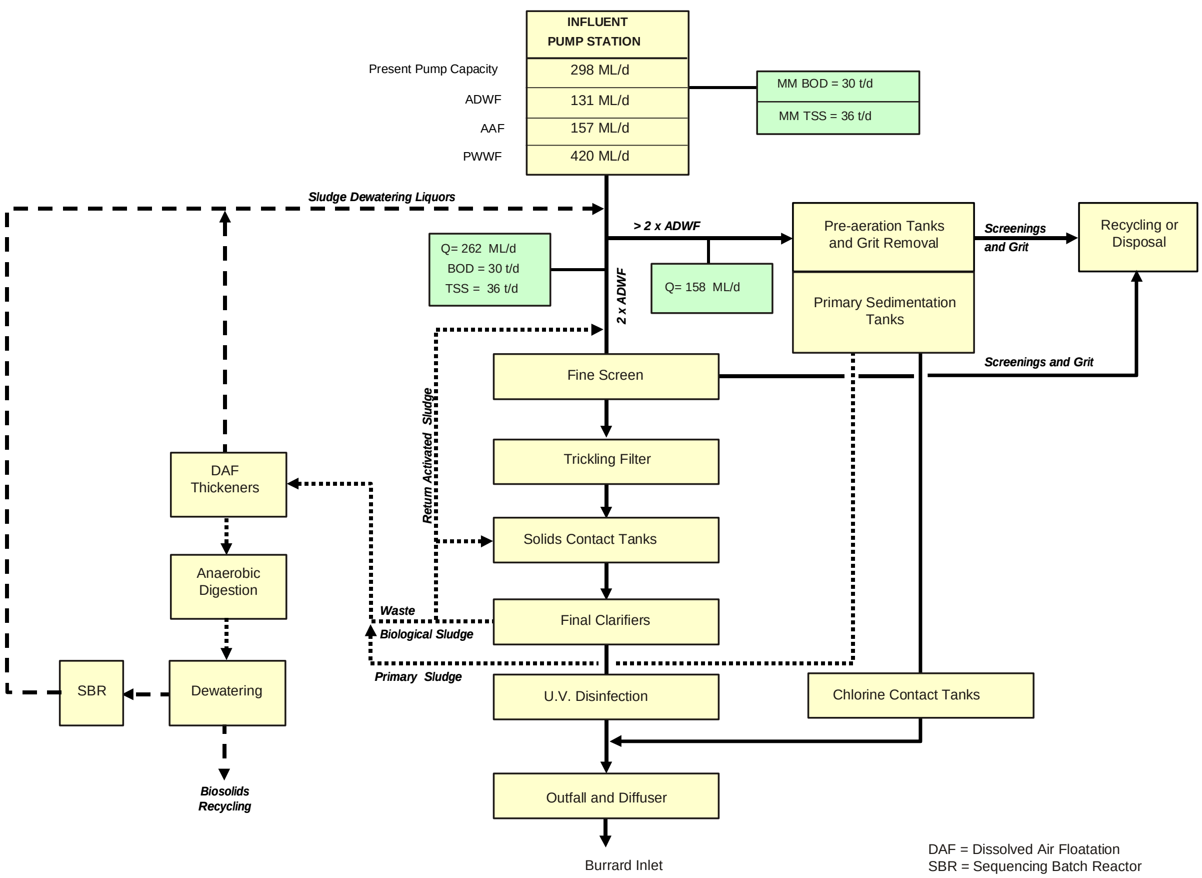


8.2.2.5 Option 5: 100% of 2 x ADWF TF/SC in parallel with Primary

When the footprint required by the HRAS plant was established, it was clear that it could not fit on the available site. The accommodation of a TF/SC plant is possible but with some difficulty. An alternative for utilizing trickling filter technology was therefore sought. Observation of the configuration and performance of the Northwest Langley WWTP, which operates without primary sedimentation, utilizing fine screens to protect the trickling filter media from clogging, indicated that this could provide a more space effective option.

A process schematic is illustrated in Figure 8.13. The TF/SC unit is designed to treat two times the average dry weather flow capacity (131 ML/d x 2 = 262 ML/d) of fine screened sewage and the associated 100% of MMF loading (30 t/d BOD and 36 t/d of TSS). Final clarifiers will be used to remove TSS and biosolids (biological sludge) generated from the TF/SC process. Secondary treated effluent will receive UV irradiation for pathogen reduction. Flows greater than two times the ADWF will, after the primary treatment, bypass the secondary treatment units and discharge directly to the chlorination system and outfall. The arrangements for solids handling and sludge treatment effluent are the same as in Option 1.

FIGURE 8.13
PROCESS SCHEMATIC OF LGWWTP UPGRADE OPTION 5



8.2.3 <u>Tank Size and Number of Units Required</u>

A spreadsheet model was developed to carry out the conceptual process design and to determine the area of units required for each unit treatment process in each upgrade option. The model summary is included in Appendix A. Unit process areas required are listed in Table 8.11.

TABLE 8.11
LGWWTP UNIT PROCESS DIMENSIONS FOR EACH UPGRADE OPTION

YEAR	2046 Build-out to Secondary				
Option	Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+TF/SC	Option 5 TF/SC + PST
Primary Clarifiers					
Existing Surface area (m2)	2,742	2,742	2,742	2,742	2,742
Depth (m)	2.75	2.75	2.75	2.75	2.75
Total Area Required (m2)	4,200	4,300	1,580	4,200	1,580
Additional Area Required (m2)	1458	1558	0	1458	0
BAF/Aeration/Solids Contact Tank					
Depth (m)	5.0	3.7	5.0	5.0	5.0
Total Area Required (m2)	1,419	1,889	3,738	852	2,215
Roughing Trickling Filter (RTF)					
Depth (m)					
Total Area Required (m2)					
Trickling Filter (TF)					
Depth (m)	6.5			4.4	8.3
Total Area Required (m2)	1,875			1,860	2,214
Final Clarifiers					
Depth (m)	4.5		4.5	4.5	4.5
Total Area Required (m2)	6,113		8,733	3,668	6,113
DAF Thickeners					
Depth (m)	3.3	3.3	3.3	3.3	3.3
Total Area Required (m2)	299.0	327.1	734.0	403.3	516.2
Digester					
Diameter (m) (Nos.5 and 6)	19.8	19.8	19.8	19.8	19.8
Depth (m)	10.1	10.1	10.1	10.1	10.1
Unit Size (m3)	3,110	3,110	3,110	3,110	3,110
Existing Volume (m3) (Nos. 3&4)	6,220	6,220	6,220	6,220	6,220
Total Volume Required (m3)	12,302	13,459	15,100	16,592	10,618
Additional Volume Required (m3)	6,082	7,239	8,880	10,372	4,398
Centrifuge					
Digested Sludge Volume (m3/d)	820	897	1007	1106	708
Existing Capacity (m3/d) - 2 No. at 35 hrs/week	540	540	540	540	540
Additional Capacity Required (m3/d)	280	357	467	566	168
Pressate Treatment (SBR)					
Depth (m)	4.5	4.5	4.5	4.5	4.5
Total Area Required (m2/d)	299	327	363	403	256

8.2.4 Conceptual Site Layout

Conceptual site layouts for each upgrade option (excluding HRAS) are illustrated in Figures 8.14, 8.15, 8.16 and 8.17. Expansion of the inlet pump station is shown adjacent to the existing pump station. Where required, the primary sedimentation tanks

(PST) are first expanded by the extension of the existing tanks 3 to 8 to the same length as tanks 1 and 2. Where required, separate additional tanks are provided. Large footprint units (bioreactors and clarifiers) are located to allow future expansion where possible. The smaller units (DAF, SBR and digesters) are located in the remaining available space along with the service buildings, such as pump stations and blower buildings. Where required, administration and operations buildings will be expanded, by increasing the height of the existing buildings. The proposed use of UV disinfection for the secondary treated effluents in build-out to secondary, obviates the need for construction of additional chlorine contact tanks. The existing chlorine contact tanks would be retained for disinfection of primary treated effluent. Chemical treatment systems are accommodated on the site of the existing digesters numbers. 1 and 2, which are to be demolished.

For comparison purposes, the approximate additional footprint requirements of each upgrade option are presented in Table 8.12. The available area, assuming that the existing plant components remain, is some 10,400 m^2. At the design treatment capacity of 262 ML/d, the HRAS option is unable to fit on the site, the TF/SC option will be difficult to fit on the site. Both the CEP plus 60% 2 x ADWF and TF/SC in parallel with primary can fit on the site. It should be noted that the BAF option can fit on the site and allow space for expansion beyond 2046. All the other options present problems in this regard.

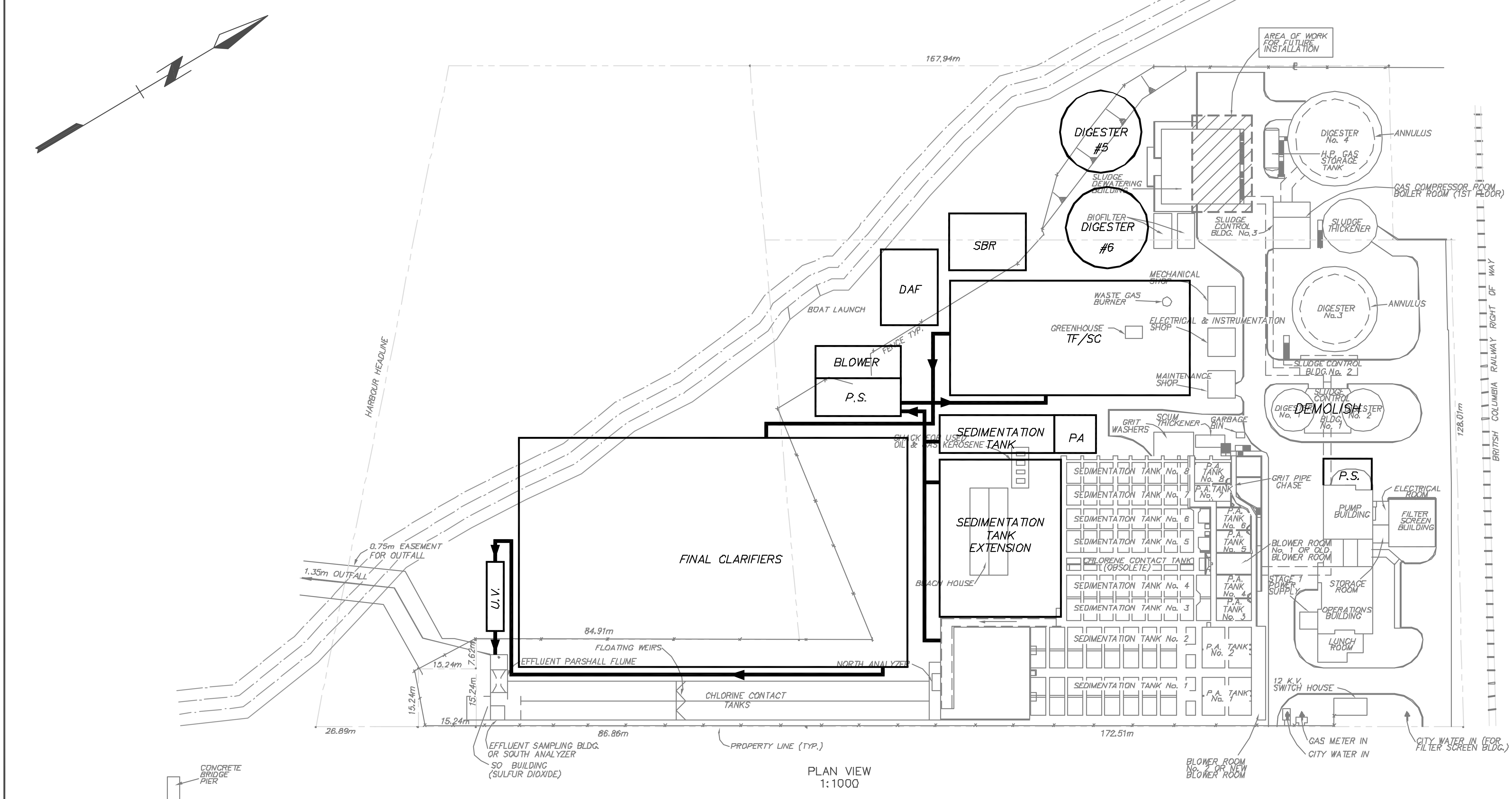
TABLE 8.12
LGWWTP FOOTPRINT REQUIREMENTS FOR EACH UPGRADE OPTION

Year	**2046 Build-out to Secondary**				
Option	Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+TF/SC	Option 5 TF/SC + Primary
Additional Footprint Required	10,384	4,652	14,120	8,391	9,076
% Use of Available Area	100%	45%	135%	81%	87%

8.2.5 Projected Effluent Quality

The projected effluent BOD and TSS concentrations of these five options are summarized in Table 8.13. At the design flow (2×ADWF), the effluent average BOD and TSS concentrations are expected to be approximately 20 mg/L. Better effluent quality can be expected at the design AAF condition. Substantial effluent toxicity reduction can be expected for all treatment options. Modeling indicates that BOD and TSS effluent concentration at peak flows will meet the effluent criteria of 45 mg/L.

FIGURE 8.14



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Stantec

DWG. No. 415-1-103

ACAD DWG. OPTION1 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	Appd	Description

0 20 Bar Is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

Design: T.W.
Drawn: KS
Checked:
Approved
Manager

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

LIONS GATE WASTEWATER TREATMENT PLANT

BUILD-OUT TO SECONDARY TREATMENT-2046

OPTION 1: PRIMARY + 2 x ADWF TF/SC

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

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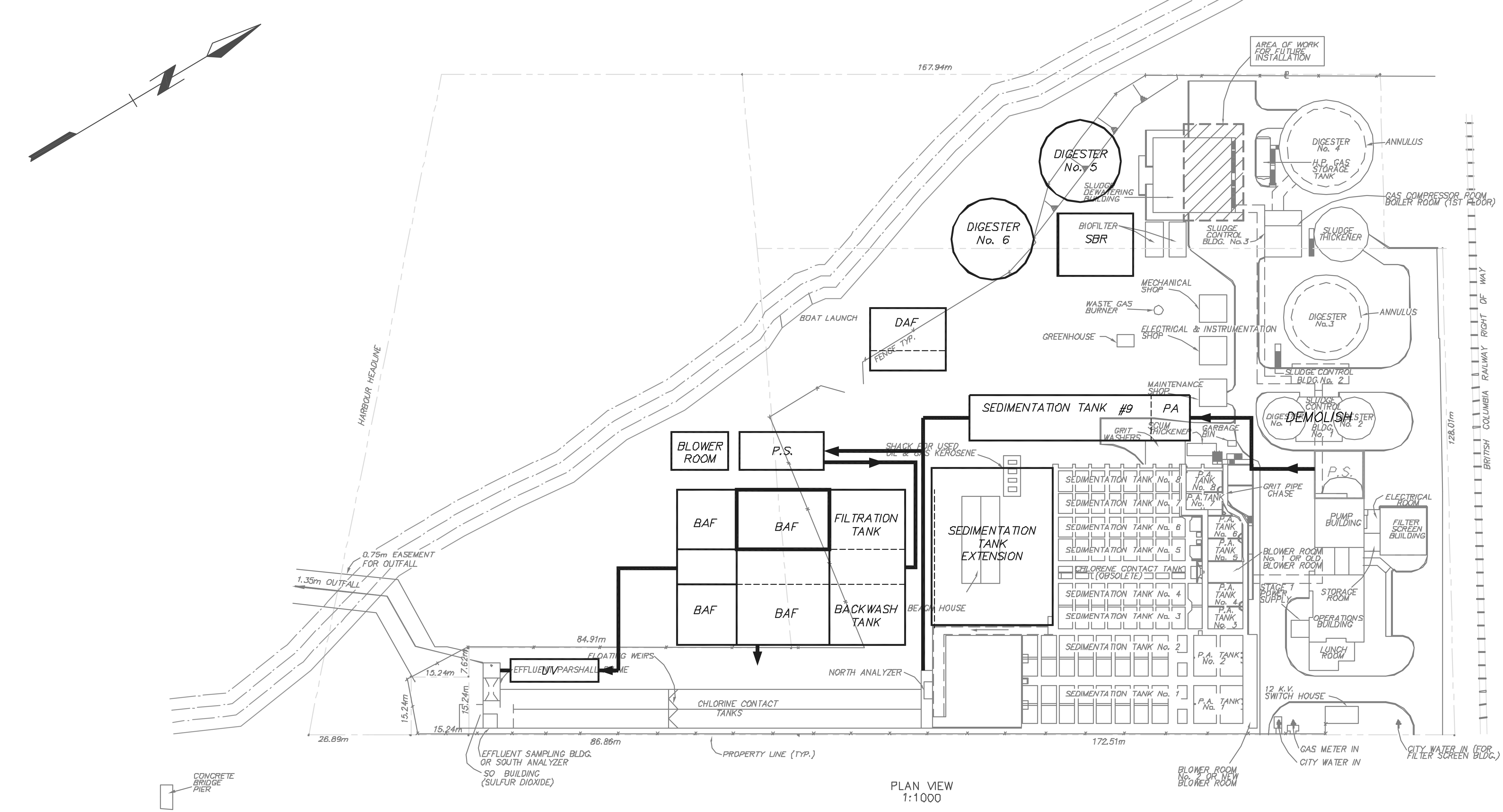
XREF DWG

SAROS DWG. ID No.

FIGURE 8.15



Dayton & Knight Ltd.

Stantec

DWG. No. 418-1-103

ACAD DWG. OPTION2 1:1 03-12-06

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	Appd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: PTL
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

BUILD-OUT TO SECONDARY TREATMENT-2046

OPTION 2: PRIMARY + 2 x ADWF BAF

SCALE: AS SHOWN

DISTRICT FILE

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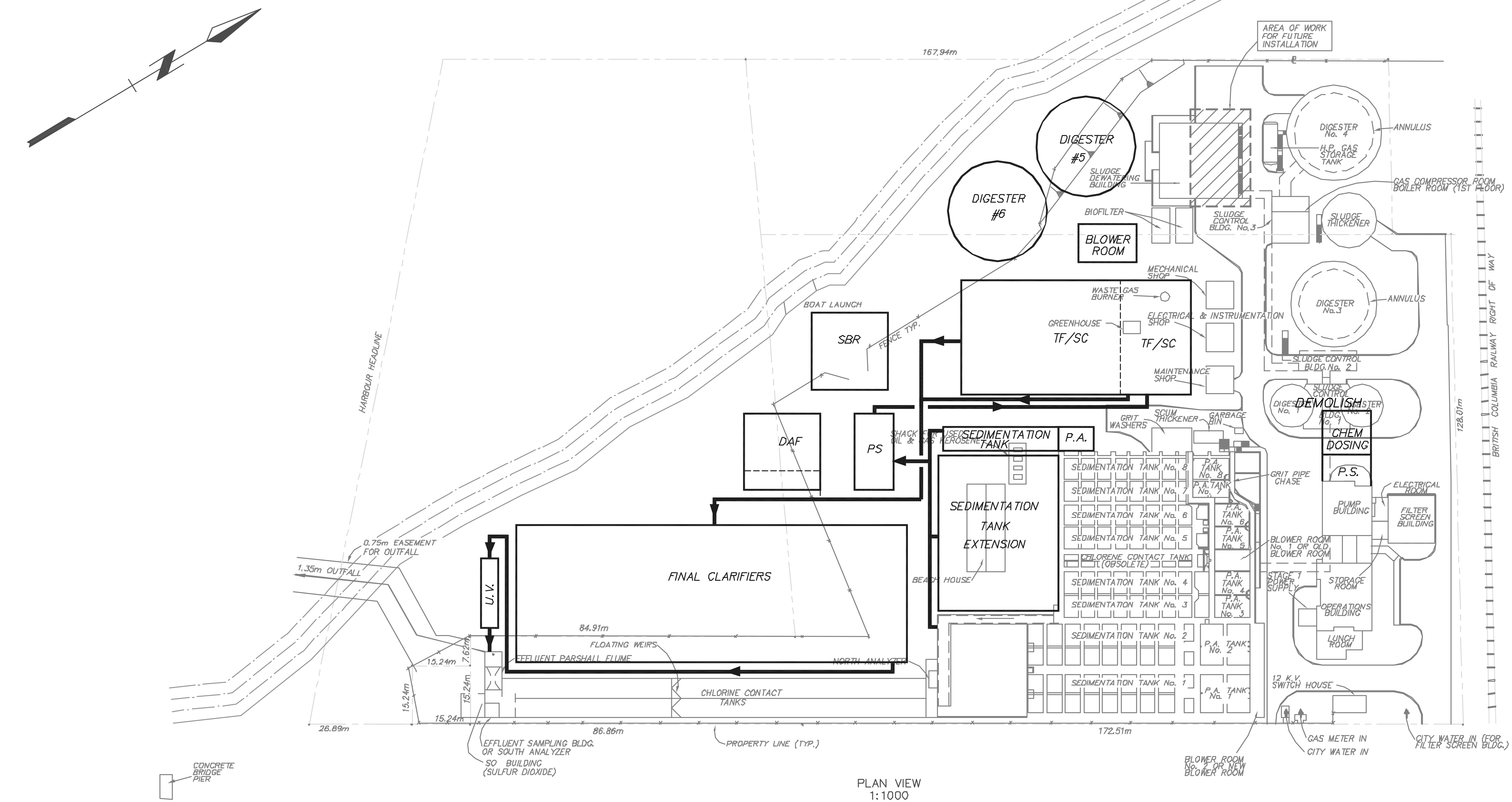
XREF DWG:

SAROS DWG. ID No.:

FIGURE 8.16



PLAN VIEW
1:1000

Dayton & Knight Ltd.

Stantec

DWG. No. 418-1-103

ACAD DWG. OPTION4 1:1 03-12-08

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

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GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: T.W.
Drawn: KS
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

BUILD-OUT TO SECONDARY TREATMENT-2046

OPTION 4: CEP + 60% 2 x ADWF TF/SC

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

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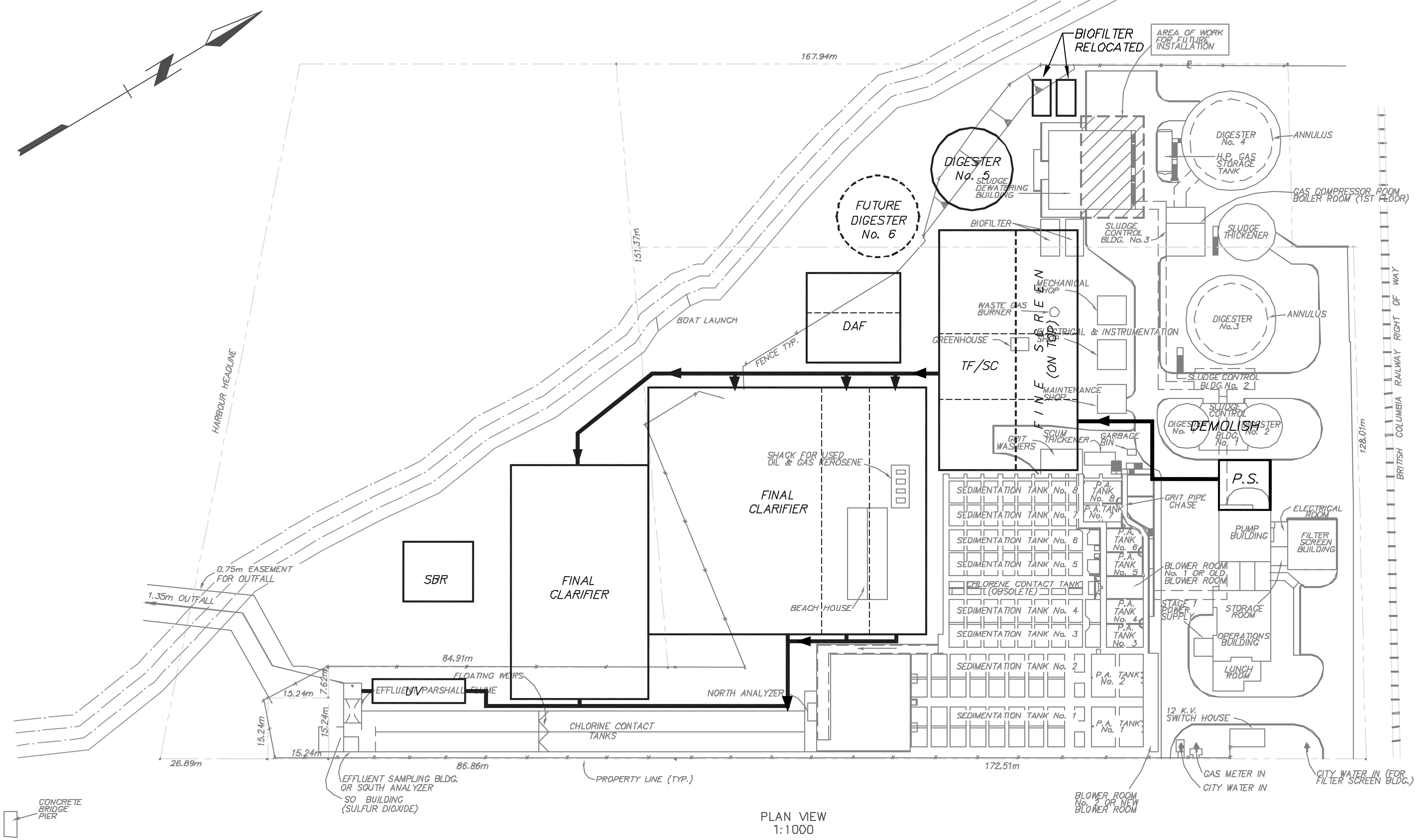
XREF DWG

SAROS DWG. ID No.

FIGURE 8.17



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Stantec

DWG. No. 418-1-103 ACAD DWG. OPTION5 1:1 03-12-05

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

Design:
Drawn: PTL
Checked:
Approved
Manager

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

LIONS GATE WASTEWATER TREATMENT PLANT

BUILD-OUT TO SECONDARY TREATMENT-2046

OPTION 5: 2 x ADWF TF/SC

IN PARALLEL WITH PRIMARY

SCALE: AS SHOWN

DISTRICT FILE

DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:

TABLE 8.13
LGWWTP EFFLUENT CONCENTRATION PROJECTIONS FOR EACH UPGRADE OPTION

YEAR	2046 Secondary to Build-out				
	Option 1	Option 2	Option 3	Option 4	Option 5
Option	TF/SC	BAF	HRAS	CEP+TF/SC	TF/SC + PST
Design AAF/2*ADWF					
BOD (mg/L)	20	20	20	20	20
SS (mg/L)	20	20	20	20	20

8.2.6 Sludge Production Projection

The projected sludge production of each upgrade option is shown in Table 8.14 for the primary sludge, CEP sludge, and biological sludge respectively. The estimated sludge volumes at each sludge handling stage is also shown in Table 8.15 including, DAF sludge, digested sludge and dewatered sludge. The sludge production increase, compared with the current production (2002 calculated annual total), is summarized in Table 8.15, based either on dry mass or volume.

Excluding HRAS, CEP + TF/SC is expected to produce the largest volume of dewatered sludge. BAF is expected to produce slightly more than TF/SC while TF/SC in parallel with primary treatment for flow greater than 2 x ADWF is expected to produce the least volume of sludge.

TABLE 8.14
SLUDGE PRODUCTION FOR EACH UPGRADE OPTION (MAXIMUM MONTH LOADS)

YEAR		2046 Secondary to Build-out				
Option	Unit	**Option 1**	**Option 2**	**Option 3**	**Option 4**	**Option 5**
		TF/SC	**BAF**	**HRAS**	**CEP+TF/SC**	**TF/SC + PST**
Raw Sludge/Biosolids						
Primary Sludge	T/d	18	18	0	-	0
CEP Sludge	T/d	0	0	0	33	0
Secondary Biosolids	T/d	11	13	35	6	25
Total Raw Sludge	T/d	29	31	35	39	25
Thickened Sludge						
Gravity Thickener (5%)	m^3/d	0	0	0	0	0
DAF (3.5%)	m^3/d	820	897	1007	1106	708
Total Thickened Sludge	m^3/d	820	897	1007	1106	708
Digested Sludge	m^3/d	820	897	1007	1106	708
Dewatered Sludge	m^3/d	72	79	99	98	67

TABLE 8.15
LGWWTP INCREASES OF SLUDGE PRODUCTION COMPARED WITH CURRENT LEVEL (MAXIMUM MONTH LOADS)

YEAR Option	Unit	2046 Secondary to Build-out				
		Option 1 TF/SC	Option 2 BAF	Option 3 HRAS	Option 4 CEP+TF/SC	Option 5 TF/SC + PST
Raw Sludge	%	123	138	96	200	91
Thickened Sludge	%	215	245	180	325	172
Digested Sludge	%	215	207	180	325	172
Dewatered Sludge	%	248	230	216	369	224

8.2.7 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 8.16. Detailed breakdowns of the process design are included in Appendix B.

Option 4 - CEP + 60% TF/SC has the lowest capital cost followed by Option 2 BAF. Option 1 TF/SC has a slightly higher capital cost than BAF while Option 5 TC/SC in parallel with Primary has a capital cost some 11% higher than TF/SC. This higher cost is related to:

- the cost of providing fine screens while leaving the primary sedimentation tanks under utilized,
- the higher cost of trickling filters to treat the greater organic load, and
- the production of only secondary sludge which results in lower unit loads on the DAF process.

Process manipulations to overcome these issues will be discussed in a subsequent section.

If the interim upgrade Option 1 (50% ADWF BAF) was selected, the treatment equipment built in the interim upgrade would be incorporated in the built-out Option 2 (BAF for 2 x ADWF). Similarly, if interim upgrade Option 2 (50% ADWF TF) or interim Option 3 (CEP + 50% ADWF RTF) were selected, the treatment equipment would be incorporated into built-out Options 1 (TF), built-out Option 4 (CEP + TF/SC) or built-out Option 5 (TF/SC + PST) and the cost of the built-out options would be reduced. However, no interim option has been selected at this time. For this reason, the cost estimates for build-out options were developed as stand alone estimates.

There are several suppliers of BAF equipment and each supplier has a unique product which is patented. The capital cost estimates are based on equipment cost provided by one supplier. There is no licensing fee as long as the equipment is purchased directly from the supplier/manufacturer.

Process units are typically shown as line items and include cost of equipment, installation, tankage, mechanical, and building cost. Excavation and process control is not included in the line items.

TABLE 8.16
LGWWTP CAPITAL COSTS OF EACH UPGRADE OPTION

YEAR	**2046 Build-out**			
Option	Option 1 TF/SC	Option 2 BAF	Option 4 CEP+TF/SC	Option 5 TF/SC + PST
CAPITAL COSTS				
Site Improvements	$5,687,000	$4,466,000	$5,090,000	$5,348,000
Chemical Dosing	$0	$0	$500,000	$0
Primary Clarifiers	$5,914,000	$6,319,000	$5,914,000	$0
Fine Screen	$0	$0	$0	$5,610,000
Trickling Filter	$12,739,000	$0	$8,184,000	$18,375,000
Bioreactor	combined with TF	$23,650,000	combined with TF	combined with TF
Final Clarifiers	$16,166,000	$0	$9,700,000	$16,166,000
Gravity Thickeners	$0	$0	$0	$0
DAF Thickeners	$6,251,000	$6,839,000	$8,430,000	$10,790,000
Digesters	$6,262,000	$7,453,000	$10,679,000	$4,528,000
Mechanical Dewatering	$2,509,000	$2,509,000	$3,763,000	$1,254,000
SBR	$0	$0	$0	$0
UV	$2,620,000	$2,620,000	$1,572,000	$2,620,000
Odour Control System	$490,000	$130,000	$370,000	$845,000
Site Works	$650,000	$2,650,000	$648,000	$581,000
Admin/Maint. Building	$2,000,000	$2,000,000	$2,000,000	$2,000,000
Control System	$2,451,000	$2,345,000	$2,274,000	$2,725,000
Electrical Substation	$85,000	$100,000	$75,000	$85,000
Existing Facility Upgrades	$0	$0	$0	$0
Division 1 Cost	$1,453,000	$1,415,000	$1,353,000	$1,639,000
Engineering	$10,212,000	$9,773,000	$9,472,000	$11,348,000
Project Management/QA/QC	$2,553,000	$2,443,000	$2,368,000	$2,837,000
Contingency	$19,147,000	$18,324,000	$17,760,000	$21,278,000
Subtotal	$97,188,000	$93,036,000	$90,152,000	$108,031,000
Net GST, 0% of Sub-Total	$0	$0	$0	$0
Total Capital Costs	$97,188,000	$93,036,000	$90,152,000	$108,031,000

8.2.8 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs for each upgrade option are shown in Table 8.17. The existing primary plant has a staff of 12 persons. For all options it is estimated that the staff would increase to 17 full-time persons. The chemical costs in Option 1, 2 and 5 are for the polymer used for dewatering uses (i.e. centrifuge). The chemicals in Option 4 include the alum and polymer used for CEP as well as the polymer needed for dewatering.

Electricity costs are based on existing costs and installed power and energy use on the plant site and the current BC Hydro, Business, Medium Power Tariff. Natural gas consumption has been based on the mass of sludge to be digested and the present cost of gas.

Maintenance has been taken as the existing cost plus a fixed %per annum (2.35%) of the total improvement capital value. Administration costs have been increased pro rata the annual average flow to the plant. No chemical disinfection (i.e. chlorination) is considered in this upgrade.

The residuals management costs are estimated based on a rate of $100/wet tonne for hauling, recycling (e.g. land application), and other fixed expenses, if application sites are available. The solids concentration is estimated to be approximately 27% for Options 1, 2 and 4 and 25% for Option 5 using mechanical dewatering by centrifuge.

TABLE 8.17
LGWWTP OPERATING AND MAINTENANCE COSTS OF EACH UPGRADE OPTION

YEAR	**2046 Build-out**			
Option	Option 1	Option 2	Option 4	Option 5
	TF/SC	BAF	CEP+TF/SC	TF/SC + PST
O&M COSTS				
Labour	$1,975,000	$1,975,000	$1,975,000	$1,975,000
Chemical Costs	$208,000	$228,000	$2,143,000	$180,000
Residuals Management	$2,639,000	$2,887,000	$3,559,000	$2,460,000
Energy	$629,000	$808,000	$653,000	$630,000
Repair/Maintenance	$2,108,000	$2,074,000	$2,051,000	$2,194,000
Administration and others	$809,000	$806,000	$803,000	$818,000
Land and building Lease	$332,000	$332,000	$332,000	$332,000
Total (O&M Costs)	$8,699,000	$9,109,000	$11,516,000	$8,588,000

8.2.9 Life Cycle Cost Estimates

The preliminary life cycle cost (LCC) with a base date of November 2003, of each upgrade option is presented in Table 8.18, using a 6% real discount rate with 30 years as the evaluation period (commencing 2032). Assuming that construction will commence in 2028 and be completed in 2030; Option 1 - TF/SC, Option 2 - BAF and Option 5 – TF/SC + PST offer the lowest costs. Option 4 - CEP with 60% TF/SC treatment has the highest cost.

TABLE 8.18
LGWWTP LIFE CYCLE COST ESTIMATE OF EACH UPGRADE OPTION

YEAR	**2046 Build-out**			
Option	Option 1	Option 2	Option 4	Option 5
Total 25-yr. O&M Costs	$23,581,000	$24,693,000	$31,217,000	$23,281,000
Discounted Capital Costs	$21,387,000	$20,473,000	$19,839,000	$23,773,000
Total Capital and O & M Costs at present value	$44,969,000	$45,166,000	$51,056,000	$47,055,000

8.2.10 Flexibility of Phasing

The plant development can be facilitated by phasing in modular expansions from interim to build-out to secondary. In order to transition from RTF interim to TF/SC secondary treatment, it will be necessary to build the solids contact tank at the interim stage. This is because, in order to reduce the footprint, the trickling filter must be constructed above the solids contact tank. The aeration/mixing system would not be included at this time. Increasing the height of the RTF units will allow them to be included as part of the build-out to secondary using the TF/SC process. Additional sludge handling units can be phased in when needed. All capital investment during the interim stage can be used in the build-out to secondary stage.

8.2.11 Energy Requirement

The major energy requirement for operating the secondary processes is in pumping and aeration. The existing influent pumps are still necessary to elevate the flow into the plant. Additional energy is required to raise the flow into the bioreactors (trickling filter, BAF or activated sludge) from where it flows by gravity to the final clarifier, and/or disinfection system and to the effluent outfall. Additional energy is required in all options to serve internal recycling (e.g. filter backwash, return activated sludge) and sludge handling operation (e.g. waste activated sludge, scum collection, UV disinfection, DAF, anaerobic digesters, dewatering).

Aeration power is essential to the BAF, TF/SC and SBR bioreactors. Ventilation and odour control is also required in the fine screening, TF/SC and anaerobic digester operation. Heat energy such as natural gas is needed to operate the digester at mesophilic or thermophilic conditions. The energy requirements of each upgrade option are presented in Table 8.19.

TABLE 8.19
LGWWTP ENERGY REQUIREMENT OF EACH UPGRADE OPTION

YEAR	**2046 Build-out**			
Option	Option 1	Option 2	Option 4	Option 5
	TF/SC	BAF	CEP+TF/SC	TF/SC + PST
Energy Requirement				
Electricity, kWh/yr	9,220,000	12,540,000	8,760,000	9,600,000
Natural Gas, GJ/yr	15,300	16,500	19,500	13,600
Electricity Cost	$461,000	$627,000	$438,000	$480,000
Natural Gas Cost	$168,000	$181,000	$215,000	$150,000
Total Energy Cost	$629,000	$808,000	$653,000	$630,000

8.2.12 Ability to Handle Load Variability

The maximum flows to the biological treatment are fixed at 262 ML/d (200% of ADWF). The excess flow will bypass secondary treatment, therefore these biological processes will not be hydraulically overloaded under normal operating conditions. The ability to increase the hydraulic load on the BAF is limited as it is operating near to its design limit. The hydraulic load on the TF/SC can be increased significantly and will be limited by the pumping capacity. CEP can be set up to adjust the chemical dosing automatically in accordance with the flow variation.

The design loads on the biological process are based on the MML and will be highest when the concentrations of BOD and TSS in the feed are highest. This is expected to occur during dry spells. The ability to increase the organic load on the BAF is limited, as this requires more frequent backwashing, which results in reduced capacity as a consequence of reduced treatment time. The organic load on the TF/SC can be significantly increased, all-be-it with resulting reduction in SBOD removal efficiency. CEP cannot be easily adjusted to meet variations in the load unless the necessary real-time monitoring equipment is installed. However the downstream biological process could absorb some of the load fluctuation. The combination of CEP and BAF has not been well proven. In view of the small footprint required by BAF, the motivation for this combination needs to be confirmed.

Apart from BAF, the space requirements for nitrification and biological nutrient removal would appear to be precluded by the lack of available space on the existing site.

8.2.13 Visual Impact

The BAF option will not cause any more adverse visual impact than the existing primary plant. The TF/SC options will impose a visual impact similar to the Annacis Island and Lulu Island WWTPs. The CEP process will not have a visual impact. Sludge handling will have an impact similar to the existing digesters.

9 SECOND LEVEL OF SCREENING

Details of the screening process are set out in Section 10 of Appendix 3. The options selected for final consideration are set out below.

Iona Island Build Out to Secondary Treatment

- TF/SC
- BAF

Iona Island interim Treatment

- 50% RTF
- CEP + 50% RTF (no secondary clarifier)
- Together these allow the interpolation of any level of CEP
- In addition to the above, the option to upgrade existing processes to meet the existing effluent standards under the increasing loads should be assessed

Lions Gate Build-out to Secondary Treatment

- BAF

Lions Gate Interim Treatment

- 50% BAF
- CEP + 50% BAF
- Together these allow the interpolation of any level of CEP
- In addition to the above, the option to upgrade the existing processes to meet the existing effluent standards up to 2031, should be assessed

The results of these further evaluations are presented in Appendix 10 Report.

APPENDIX A: PROCESS DESIGN SUMMARY

IIWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Average Dry Weather Flow (ML/d), ADWF	500	500	616	500
Average Annual Flow (ML/d), AAF	650	650	789	650
Peak Wet Weather Flow (ML/d), PWWF	1530	1530	1908	1530
Maximum Month BOD Loading (t/d), MM BOD	124	124	150	124
Maximum Month TSS Loading (t/d), MM TSS	113	113	144	113
Primary Clarifier				
Average Annual Flow (ML/d)	650	650	789	650
Peak Wet Weather Flow (ML/d)	1530	1530	1908	1530
Overflow Rate ($m^3/m^2/d$) - AAF	40	40	40	40
Overflow Rate ($m^3/m^2/d$) - PWWF	100	100	100	100
Surface Area (m^2) - AAF	16250	16250	19725	16250
Surface Area (m^2) - PWWF	15300	15300	19080	15300
Depth (m)	2.74	2.74	2.74	2.74
Volume (m^3) - AAF	44525	44525	54046.5	44525
Volume (m^3) - PWWF	41922	41922	52279.2	41922
Raw Influent BOD Loading (t/d)	124	124	150	124
Raw Influent TSS Loading (t/d)	113	113	144	113
Total Influent BOD Loading (t/d)	124	124	150	124
Total Influent TSS Loading (t/d)	113	113	144	113
Design PC BOD removal (%)	35%	35%	35%	55%
Design PC TSS removal (%)	50%	50%	50%	80%
PC Effluent BOD Loading (t/d)	80.6	80.6	97.5	55.8
PC Effluent TSS Loading (t/d)	56.5	56.5	72	22.6
PC Effluent BOD Conc. @ AAF (mg/L)	124	124	124	86
PC Effluent TSS Conc. @ AAF (mg/L)	87	87	91	35
Chemical Usage	N/A	N/A	N/A	
Alum Dosage (mg/L)				70
Polymer Dosage (mg/L)				0.5
Alum Volume - AAF (m^3/d)				69.29637527
Polymer Volume - AAF (m^3/d)				0.8125
Alum Volume - PWWF (m^3/d)				163.1130064
Polymer Volume - PWWF(m^3/d)				1.9125
Biological Treatment				
Treating % of ADWF	200%	200%	200%	120%
Design Flow (ML/d)	1000	1000	1232	600
Treating % of MM BOD loading	100%	100%	100%	60%
Design BOD Loading (t/d)	80.6	80.6	97.5	33.48
Aeration Basin	N/A			
Design MLSS (mg/L) CAS or HRAS		2000	2000	2000
MLVSS/MLSS		0.8	0.8	0.8
Design F/M (kg BOD/kg MLVSS)		0.25	0.25	0.21
Sludge Yield		0.7	0.7	0.7
Solids Retention Time SRT (days)		6	6	6
Aeration Basin Volume (m^3)		201500	243750	99642.85714
Surface Area Required (m^2)		40300	48750	19928.57143
Oxygen Requirement (kg O_2/kg BOD_5)		1.2	1.2	1.2
Actual Oxygen Transfer Rate AOTR (t/d O_2)		96.72	117	40.176
SOTR (t/d O_2)		213	257	88
Air requirement (scfm)		63835	77220	26516
Design Effluent BOD Concentration (mg/L)		20	20	20

IIWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
TF/SC		N/A	N/A	N/A
Design Trickling Filter Loading (kg BOD/m^3/d)	1.6			
Volume of Trickling Filter (m^3) - organic load	50375			
Depth of Tower (m)	6			
Area of Trickling Filter (m^2)	8396			
Design AAF Hydraulic Loading m^3/m^2/d-Minimum	45			
Average Hydraulic Loading rate (m^3/m^2.d)	60			
Design MLSS (mg/L) in SC	2000			
MLVSS/MLSS	0.8			
Design F/M (kg BOD/kg MLVSS)	0.28			
Observed Sludge Yield	0.71			
Effective "Solids Retention Time" (days)	6			
Aeration Basin Volume (m^3)-F/M ratio	44438			
HRT (hr) @ Flow AAF	1.6			
Aeration Basin Depth (m)	4.5			
Surface Area Required (m^2) - F/M ratio	9875			
Minimum HRT Requirement (hr)	0.65			
Aeration Basin Volume (m^3)- HRT	27083			
Surface Area Required (m^2) - HRT	6019			
Return Activated Sludge % (RAS)	75%			
Air requirement (scfm)	15046			
Design Effluent BOD Concentration (mg/L)	20			
Final Clarifier				
Surface Overflow Rate (m^3/m^2/d) -Max flow	60	45	45	45
Surface Area 1 (m^2) -SOR	16667	22222	27378	13333
Solids Loading Rate (kg/m^2/d)-Max Flow	150	150	150	150
Surface Area 2 (m^2) -SLR	23333	23333	28747	14000
Depth (m)	4.5	4.5	4.5	4.5
Volume (m^3)	105000	105000	129360	63000
HRT (hr) @ Design Flow	1.44	1.44	1.44	1.44
Design Effluent TSS Concentration (mg/L)	20	20	20	20
Thickener - Gravity (for PS)				
Raw Primary Sludge (t/d)	56.5	56.5	72	90.4
Chemical Sludge (t/d)				15
Total Primary/CEP Sludge (t/d)	56.5	56.5	72	106
Solids Concentration After Thickening (%)	5%	5%	5%	5%
Sludge Volume (m^3/d)	1130	1130	1440	2111
Design Solids Loading (kg/m^2/d)	100	100	100	100
Surface Area (m^2)	565	565	720	1056
Thickener - DAF (for WAS)				
Waste Activated Sludge (t/d) WAS	47.996	47.32	57.204	14.336
Solids Concentration After DAF (%)	0.035	0.035	0.035	0.035
Sludge Volume (m^3/d)	1371	1352	1634	410
Design Solids Loading (kg/m^2/d)	48	48	48	48
Surface Area (m^2)	1000	986	1192	299

IIWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2036 Build-out to Secondary**			
Option	Option 1 Primary +100% 2* ADWF TF/SC	Option 2A Primary + 100% 2* ADWF CAS	Option 2B Primary + 100% 2* ADWF CAS (VSA and NSSA)	Option 3 CEP + 60% 2*ADWF CAS
Digester (Thermophilic Anaerobic)				
Digester VSS Loading (kg/d/m^3)	2.5	2.5	2.5	2.5
Sludge VSS/TSS Ratio	0.78	0.78	0.78	0.78
Digester Volume (m^3) by VSS loading	32603	32392	40312	37410
Un-digested dry tonnes (T/d)	104	104	129	120
Digested dry tonnes (T/d)	72	71	89	82
Design HRT (d)	15	15	15	15
Digested Sludge Solids (%)	2.9%	2.9%	2.9%	3.3%
VS destruction %	40%	40%	40%	40%
Digested sludge VSS (T/d)	48.9	48.6	60.5	56.1
Digested sludge VSS/TSS ratio	0.68	0.68	0.68	0.68
Digested Sludge Volume (m^3/d) (without dewatering)	2501	2482	3074	2521
Digester Volume (m^3) by Design HRT	37520	37230	46116	37814
Dewatering				
Centrifuge (L/min)	1200	1200	1200	900
Days of Operation / week	5	5	5	5
Hours of operation / day	7	7	7	7
Sludge Cake (%) (dewatered)	27%	27%	27%	27%
Sludge Cake (m3/d) (dewatered)	266	265	329	306
Estimated Effluent @ 2 x ADWF				
BOD (mg/L)	20	20	20	20
SS (mg/L)	20	20	20	20

LGWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN

YEAR	**2046 Secondary Winter Conditions**				
	Option 1	Option 2	Option 3	Option 4	Option 5
	TF/SC	BAF	HRAS	CEP+TF/SC	TF/SC + PST
Average Dry Weather Flow (ML/d), ADWF	131	131	131	131	131
Average Annual Flow (ML/d), AAF	157	157	157	157	157
Peak Wet Weather Flow (ML/d), PWWF	420	420	420	420	420
Maximum Month BOD Loading (t/d), MM BOD	30	30	30	30	30
Maximum Month TSS Loading (t/d), MM TSS	36	36	36	36	36
Primary Clarifier					
Average Annual Flow (ML/d)	157	157	5	157	5
Peak Wet Weather Flow (ML/d)	420	420	158	420	158
Overflow Rate ($m^3/m^2/d$) - AAF	40	40	40	40	40
Overflow Rate ($m^3/m^2/d$) - PWWF	100	100	100	100	100
Surface Area (m^2) - AAF	3,925	4,175	125	3,925	125
Surface Area (m^2) - PWWF	4,200	4,300	1,580	4,200	1,580
Depth (m)	2.79	2.79	2.79	2.79	2.79
Volume (m^3) - AAF	10,951	11,648	349	10,951	349
Volume (m^3) - PWWF	11,718	11,997	4,408	11,718	4,408
Raw Influent BOD Loading (t/d)	30	30	1	30	1
Raw Influent TSS Loading (t/d)	36	36	1	36	1
Total Influent BOD Loading (t/d)	30	30	1	30	1
Total Influent TSS Loading (t/d)	36	36	1	36	1
PC Influent MM BOD Conc. @ AAF (mg/L)	191	191	191	191	191
PC Influent MM TSS Conc. @ AAF (mg/L)	229	229	229	229	229
Design PC BOD removal (%)	35%	35%	35%	55%	35%
Design PC TSS removal (%)	50%	50%	50%	80%	50%
PC Effluent BOD Loading (t/d)	20	20	1	14	1
PC Effluent TSS Loading (t/d)	18	18	1	7	1
PC Effluent BOD Conc. @ AAF (mg/L)	124	124	124	86	124
PC Effluent TSS Conc. @ AAF(mg/L)	115	115	115	46	115
Chemical Usage	**N/A**	**N/A**	**N/A**		**N/A**
Alum Dosage (mg/L)				70	
Polymer Dosage (mg/L)				0.5	
Alum($Al_2(SO_4)_3$) Volume - AAF (m^3/d)				16.7	
Polymer Volume - AAF (m^3/d)				0.2	
Alum($Al_2(SO_4)_3$) Volume - PWWF (m^3/d)				44.8	
Polymer Volume - PWWF(m^3/d)				0.5	
Biological Treatment					
Plant Capacity% of ADWF	200%	200%	200%	120%	200%
Design Flow (MLD)	262	262	262	157	262
Treating % of MM BOD / TSS loading	100%	100%	100%	97%	100%
Design BOD Loading (t/d)	19.5	19.5	30.0	13.1	30.0
Aeration Basin	**N/A**	**N/A**		**N/A**	**N/A**
Design MLSS (mg/L)			2,000		
MLVSS/MLSS			0.75		
Design F/M (kg BOD/kg MLVSS)			1.07		
Observed Sludge Yield			1.26		
Solids Retention Time (days)			1		
Aeration Basin Volume (m^3)			18,692		
HRT (hr) @ Design Flow			1.7		
Aeration Basin Depth (m)			5		
Surface Area Required (m^2)			3,738		
Return Activated Sludge % (RAS)			150%		
Oxygen Requirement (kg O_2/kg BOD_5)			0.85		
Actual Oxygen Transfer Rate AOTR (t/d)			21.0		
SOTR (t/d O2)			46.3		
Air requirement (sCFM)			13890		
Design Effluent BOD Concentration (mg/L)			20		

LGWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN (Cont'd)

YEAR	**2046 Secondary Winter Conditions**				
	Option 1	Option 2	Option 3	Option 4	Option 5
	TF/SC	BAF	HRAS	CEP+TF/SC	TF/SC + PST
BAF	**N/A**		**N/A**	**N/A**	**N/A**
Sludge Yield		0.94			
Sludge Age (days)		2			
Design Organic Loading (kg/m^3/d)		4.30			
Design Hydraulic Loading m^3/m^2/d-average		144			
Design Hydraulic Loading m^3/m^2/d-peak		240			
Backwash flow Ml/d		10			
Reactor Area (m^2) - organic load		1226			
Reactor Area (m^2) - average flow		1889			
Reactor Area (m^2) - peak flow		1133			
Depth (m)		3.7			
Volume Required (m^3)		6989			
Oxygen Requirement (kg O_2/kg BOD_5)		1.6			
Actual Oxygen Transfer Rate AOTR (t/d)		22.8			
SOTR (t/d O2)		50.2			
Air requirement (sCFM)		15059			
Design Effluent BOD Concentration (mg/L)		20			
Design Effluent TSS Concentration (mg/L)		20			
TF/SC		**N/A**	**N/A**		
Design Trickling Filter Loading (kg BOD/m^3/d)	1.6			1.6	1.6
Volume of Trickling Filter (m^3) - organic load	12,188			8,184	18,375
Depth of Tower (m)	6.5			4.4	8.30
Area of Trickling Filter (m^2)	1,875			1,860	2,214
Design AAF Hydraulic Loading m^3/m^2/d-Minimum	45			45	45
Average Hydraulic Loading rate (m^3/m^2.d)	70			70	59
Design MLSS (mg/L)	2000			2000	2000
MLVSS/MLSS	0.80			0.80	0.80
Design F/M (kg BOD/kg MLVSS)	0.28			0.28	0.23
Observed Sludge Yield	0.71			0.71	0.93
Effective "Solids Retention Time" (days)	6			6	6
Aeration Basin Volume (m^3) sBOD Load	5,913			3,971	11,075
HRT (hr) @ AAF	0.9			0.6	1.7
Aeration Basin Depth (m)	5.0			5.0	5.0
Foot Print Area Required (m^2) BOD	1,183			794	2,215
Minimum HRT Requirement (hr)	0.65			0.65	0.65
Aeration Basin Volume (m^3)- HRT	7,096			4,258	7,096
Surface Area Required (m^2) - HRT	1,419			852	1,419
Return Activated Sludge % (RAS)	75%			75%	75%
Oxygen Requirement (kg O_2/kg BOD_5)	1.3			1.3	1.3
Actual Oxygen Transfer Rate AOTR (t/d)	-0.5			0.1	2.7
SOTR (t/d O_2)	-1.2			0.3	6.0
Air requirement (sCFM)	-352			75	1805
Mixing requirement (m^3 air/m^3/min)	0.015			0.015	0.015
Air requirement (sCFM)	3942			2365	3942
Oxygen Transfer Efficiency %	5%			5%	5%
Peak factor	1			1	1
Oxygen Applied (kgO2/kgBOD applied)	18.28			18.28	18.28
Air flow rate at 20oC and 1.0Atm (m3/min)	1479			614	1479
Design Effluent BOD Concentration (mg/L)	20			20	20

LGWWTP BUILD-OUT TO SECONDARY UPGRADE PROCESS DESIGN (Cont'd)

YEAR	2046 Secondary Winter Conditions				
	Option 1	Option 2	Option 3	Option 4	Option 5
	TF/SC	BAF	HRAS	CEP+TF/SC	TF/SC + PST
Final Clarifier		**N/A**			
Surface Overflow Rate ($m^3/m^2/d$) -Max flow	60		45	60	60
Surface Area 1 (m^2) -SOR	4,367		5,822	2,620	4,367
Solids Loading Rate ($kg/m^2/d$)-Max Flow	150		150	150	150
Surface Area 2 (m^2) -SLR	6,113		8,733	3,668	6,113
Depth (m)	4.5		4.5	4.5	4.5
Volume (m^3) (with larger surface area)	27,510		39,300	16,506	27,510
HRT (hr) @ Design Flow + RAS	1.44		1.44	1.44	1.44
Design Effluent TSS Concentration (mg/L)	20		20	20	20
Thickener - Gravity (for PS) insert Y or N	N	N	N	N	N
Raw Primary Sludge (t/d)					
Chemical Sludge (t/d)					
Total Primary/CEP Sludge (t/d)					
Solids Concentration After Thickening (%)	5%	5.0%	5%	5%	5%
Sludge Volume (m^3/d)					
Design Solids Loading MML ($kg/m^2/d$)	100	100	100	100	100
Surface Area (m^2)					
Thickener - DAF (for WAS or Combined Primary)	Co-DAF	**Co-DAF**	**Co-DAF**	**Co-DAF**	**Co-DAF**
Sludge (t/d)	28.7	31.4	35.2	38.7	24.8
Solids Concentration After DAF (%)	3.5%	3.5%	3.5%	3.5%	3.5%
Sludge Volume (m^3/d)	820	897	1007	1106	708
Design Solids Loading ($kg/m^2/d$) 48 for WAS 96 for C	96.0	96.0	48.0	96.0	48.0
Surface Area (m^2)	299	327	734	403	516
Digester					
Digester VSS Loading ($kg/d/m^3$)	2.2	2.2	2.2	2.2	2.2
Sludge VSS/TSS Ratio	80%	80%	75%	80%	80%
Digester Volume (m^3) by VSS loading	10,438	11,420	12,011	14,078	9,009
Un-digested dry tonne (T/d)	29	31	35	39	25
Digested dry tonne (T/d)	20	21	25	26	17
Design HRT (d)	15	15	15	15	15
Digested Sludge Solids (%)	2.4%	2.4%	2.5%	2.4%	2.4%
VS destruction %	40%	40%	40%	40%	40%
Digested Sludge Volume (m^3/d)	820	897	1,007	1,106	708
Actual HRT (d)	13	13	12	13	13
Digester Volume (m^3) by Design HRT	12,302	13,459	15,100	16,592	10,618
Required Volume (Max) m^3	12,302	13,459	15,100	16,592	10,618
Dewatering					
Centrifuge (L/min)	900	900	900	900	900
Days of Operation / week	5	5	5	5	5
Hours of operation / day	7	7	7	7	7
No. Centrifuges	3.0	3.3	3.7	4.1	2.6
Sludge Cake (%) (dewatered)	27%	27%	25%	27%	25%
Sludge Cake (m3/d) (dewatered)	72	79	99	98	67
Pressate Treatment SBR					
Pressate volume (m3/d)	748	818	908	1,009	640
SBR Volume (m3/d) = 1.8 x Pressate vol.	1,346	1,473	1,634	1,816	1,153
Depth of Reactor (m)	4.5	4.5	4.5	4.5	4.5
Area of Reactor (m2/d)	299	327	363	403	256
Effluent Standard					
BOD mg/l	45	45	45	45	45
TSS mg/l	45	45	45	45	45
Estimated Effluent @ AAF/ 2*ADWF					
BOD (mg/L)	20	20	20	20	20
SS (mg/L)	20	20	20	20	20

APPENDIX B: CAPITAL COST ESTIMATES

IIWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 1, PRIMARY + 100% OF 2 X ADWF TF/SC

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			585000	m^3	$10	$5,850,000
Preloading (1.5 m & 4 m)			200000	m^3	$15	$3,000,000
Ground densification - new			1630000	m^3	$8	$13,040,000
Ground densification - exist			168000	m^3	$8	$1,344,000
Soil anchors - existing			2220	each	$4,000	$8,880,000
Dewatering				l.s.		$3,700,000
Total for Site Improvement						$43,814,000
Primary Sedimentation Tank	0	990	0	m^2	$3,630	$0
Trickling Filters	6	9123	54738	m^3	$920	$50,358,960
Solids Contact	3	14190	42570	m^3	$360	$15,325,200
Secondary Clarifiers	16	1520	24320	m^2	$2,140	$52,044,800
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	3.2	314	1004.8	m^2	$21,200	$21,301,760
Digesters	2.1	8520	17892	m^3	$940	$16,818,480
Mechanical Dewatering				l.s.		$9,960,000
Site Works:						
Pumping to Bioreactor			2080	HP	$3,000	$6,240,000
Roads/grading				l.s.		$1,000,000
750 mm RAS			2800	m	$500	$1,400,000
600 mm WAS			3640	m	$450	$1,638,000
2400 mm effluent			900	m	$1,925	$1,732,500
Admin/Maint Building	1	5000	5000	m^2	$1,600	$8,000,000
Control System (allowance)		4%		l.s.		$6,233,968
Electrical substation (allow)				l.s.		$1,500,000
Odour Control				l.s.		$1,000,000
Existing Facility Upgrades						$0
Sub-Total						**$238,367,668**
Division 1 Cost		2.5%				$4,863,842
Engineering		16%				$38,138,827
Project Management/ Quality Control		4%				$9,534,707
Contingency		30%				$71,510,300
Sub-total						**$362,415,344**
Net GST (0%)						$0
Total (Capital Costs)						**$362,415,344**

IIWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 2A, PRIMARY + 100% OF 2 X ADWF CAS

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			650000	m^3	$10	$6,500,000
Preloading (1.5 m & 4 m)			222000	m^3	$15	$3,330,000
Ground densification - new			1811000	m^3	$8	$14,488,000
Ground densification - exist			168000	m^3	$8	$1,344,000
Soil anchors - existing			2220	each	$4,000	$8,880,000
Dewatering				l.s.		$3,700,000
Total for Site Improvement						$46,242,000
Primary Sedimentation Tank	0	990	0	m^2	$3,650	$0
Aeration Basin	16	14190	227040	m^3	$360	$81,734,400
Secondary Clarifiers	16	1520	24320	m^2	$2,140	$52,044,800
Gravity Thickeners	0	308	0	m^2	$4,500	$0
DAF Thickeners	3.1	314	973.4	m^2	$21,200	$20,636,080
Digesters	2	8520	17040	m^3	$940	$16,017,600
Mechanical Dewatering				l.s.		$9,960,000
Site Works:						
Pumping to Bioreactor			1440	HP	$3,750	$5,400,000
Roads/grading				l.s.		$1,000,000
750 mm RAS			2800	m	$500	$1,400,000
600 mm WAS			3640	m	$450	$1,638,000
2400 mm effluent			900	m	$1,925	$1,732,500
Admin/Maint Building	1	5000	5000	m^2	$1,600	$8,000,000
Control System (allowance)		4%		l.s.		$7,215,715
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$254,521,095**
Division 1 Cost		2.5%				$5,206,977
Engineering		16%				$40,723,375
Project Management/ Quality Control		4%				$10,180,844
Contingency		30%				$76,356,329
Sub-total						**$386,988,620**
Net GST (0%)						$0
Total (Capital Costs)						**$386,988,620**

IIWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 2B, PRIMARY + 100% OF 2 X ADWF CAS (+ LGWWTP)

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			812500	m^3	$10	$8,125,000
Preloading (1.5 m & 4 m)			265000	m^3	$15	$3,975,000
Ground densification - new			2264000	m^3	$8	$18,112,000
Ground densification - exist			168000	m^3	$8	$1,344,000
Soil anchors - existing			2220	each	$4,000	$8,880,000
Dewatering				l.s.		$4,625,000
Total for Site Improvement						$53,061,000
Primary Sedimentation Tank	0	990	0	m^2	$3,650	$0
Aeration Basin	19	14190	269610	m^3	$360	$97,059,600
Secondary Clarifiers	19	1520	28880	m^2	$2,140	$61,803,200
Gravity Thickeners	1	308	308	m^2	$4,500	$1,386,000
DAF Thickeners	3.8	314	1193.2	m^2	$21,200	$25,295,840
Digesters	3.1	8520	26412	m^3	$940	$24,827,280
Mechanical Dewatering				l.s.		$12,800,000
Site Works:						
Pumping to Bioreactor			1780	HP	$3,250	$5,785,000
Roads/grading				l.s.		$1,100,000
750 mm RAS			3500	m	$500	$1,750,000
600 mm WAS			4550	m	$450	$2,047,500
2400 mm effluent			1100	m	$1,925	$2,117,500
Admin/Maint Building	1	5000	5000	m^2	$1,600	$8,000,000
Control System (allowance)		4%		l.s.		$8,926,877
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$307,459,797**
Division 1 Cost		2.5%				$6,359,970
Engineering		16%				$49,193,567
Project Management/ Quality Control		4%				$12,298,392
Contingency		30%				$92,237,939
Sub-total						**$467,549,665**
Net GST (0%)						$0
Total (Capital Costs)						**$467,549,665**

IIWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 3, CEP + 60% OF 2 X ADWF CAS

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			417000	m^3	$10	$4,170,000
Preloading (1.5 m & 4 m)			156000	m^3	$15	$2,340,000
Ground densification - new			1167000	m^3	$8	$9,336,000
Ground densification - exist			168000	m^3	$8	$1,344,000
Soil anchors - existing			2220	each	$4,000	$8,880,000
Dewatering				l.s.		$2,300,000
Total for Site Improvement						$36,370,000
Primary Sedimentation Tank	0	990	0	m^2	$3,650	$0
Aeration Basin	8	14190	113520	m^3	$360	$40,867,200
Secondary Clarifiers	10	1520	15200	m^2	$2,140	$32,528,000
Gravity Thickeners	2	308	616	m^2	$4,500	$2,772,000
DAF Thickeners	1	314	314	m^2	$21,200	$6,656,800
Digesters	2.1	8520	17892	m^3	$940	$16,818,480
Mechanical Dewatering				l.s.		$11,300,000
Chemical feed system						$1,500,000
Site Works:						
Pumping to Bioreactor			865	HP	$5,260	$4,549,900
Roads/grading				l.s.		$1,000,000
750 mm RAS			1700	m	$500	$850,000
600 mm WAS			2200	m	$450	$990,000
2400 mm effluent			540	m	$1,925	$1,039,500
Admin/Maint Building	1	5000	5000	m^2	$1,600	$8,000,000
Control System		4%		l.s.		$4,437,699
Electrical substation (allow)				l.s.		$1,500,000
Existing Facility Upgrades						$0
Sub-Total						**$171,179,579**
Division 1 Cost		2.5%				$3,370,239
Engineering		16%				$27,388,733
Project Management/ Quality Control		4%				$6,847,183
Contingency		30%				$51,353,874
Sub-total						**$260,139,608**
Net GST (0%)						$0
Total (Capital Costs)						**$260,139,608**

LGWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 1, PRIMARY + 2 X ADWF TF/SC

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	13992.96	14	195,901	m^3	$8	$1,567,212
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			9,600	m^2	$100	$960,000
Total for Site Improvement						$5,687,212
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	7.9	186	1,458	m^2	$4,056	$5,913,822
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	6.0	2042	12,188	m^3	$1,045	$12,738,750
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
Bioreactor	0.0	0	0	m^2		combined with TF
Secondary Clarifiers	7.0	871	6,113	m^2	$2,644	$16,166,306
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.6	113	299	m^2	$20,905	$6,250,698
Digesters	2.0	3110	6,082	m^3	$1,030	$6,261,884
Mechanical Dewatering (Centrifuge)	2.0			l.s.	$1,254,277	$2,508,554
SBR				m^3		$0
UV			262	ML/d PWWF	$10,000	$2,620,000
Odour Control	Allowance					$490,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1600 mm dia.)		1600	343.5	m	$1,600	$549,600
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			61,286,825	%	4.00%	$2,451,473
Electrical substation		1,282	1	l.s.	$85,000	$85,000
Existing Facility Upgrades						$0
Sub-Total						**$63,823,298**
Division 1 Cost		2.5%				$1,453,402
Engineering		16%				$10,211,728
Project Management/ Quality Control		4%				$2,552,932
Contingency		30%				$19,146,989
Sub-total						**$97,188,349**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$97,188,349**

LGWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 2, PRIMARY + 2 X ADWF BAF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	7822.56	14	109,516	m^3	$8	$876,127
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			4,300	m^2	$100	$430,000
Total for Site Improvement						$4,466,127
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	8.4	186	1,558	m^2	$4,056	$6,319,434
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
BAF			1	l.s.	$23,649,647	$23,649,647
Secondary Clarifiers	0.0	0	0	m^2	$2,644	$0
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	2.9	113	327	m^2	$20,905	$6,838,509
Digesters	2.3	3110	7,239	m^3	$1,030	$7,452,959
Mechanical Dewatering (Centrifuge)	2.0			l.s.	$1,254,277	$2,508,554
SBR				m^3		$0
UV			262	ML/d PWWF	$10,000	$2,620,000
Odour Control	Allowance					$130,000
Site Works:						
Pumping to Bioreactor			340	kW	$6,622	$2,251,463
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1600 mm dia.)		1600	186.5	m	$1,600	$298,400
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			58,635,092	%	4.00%	$2,345,404
Electrical substation		1,806	1	l.s.	$100,000	$100,000
Existing Facility Upgrades						$0
Sub-Total						**$61,080,496**
Division 1 Cost		2.5%				$1,415,359
Engineering		16%				$9,772,879
Project Management/ Quality Control		4%				$2,443,220
Contingency		30%				$18,324,149
Sub-total						**$93,036,103**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$93,036,103**

LGWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 3, 2 X ADWF HRAS + PRIMARY

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	19070.04	14	266,981	m^3	$8	$2,135,844
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering				m^2	$100	$0
Total for Site Improvement						$5,295,844
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	0.0	186	0	m^2	$4,056	$0
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	0.0	0	0	m^3	$1,000	$0
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
Bioreactor (HRAS)	10.0	375	3,738	m^2	$2,109	$7,884,793
Secondary Clarifiers	9.6	908	8,733	m^2	$2,644	$23,094,722
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	6.5	113	734	m^2	$20,905	$15,344,531
Digesters	2.9	3110	8,880	m^3	$1,030	$9,142,387
Mechanical Dewatering (Centrifuge)	2.0			l.s.	$1,254,277	$2,508,554
SBR				m^3		$0
UV			262	ML/d PWWF	$10,000	$2,620,000
Odour Control	Allowance					$490,000
Site Works:						
Pumping to Bioreactor			85	kW	$9,400	$798,953
Roads/grading			1	l.s.	$100,000	$100,000
600 mm WAS			3640	m	$450	$1,638,000
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			70,917,785	%	4.00%	$2,836,711
Electrical substation		1,395	1	l.s.	$85,000	$85,000
Existing Facility Upgrades						$0
Sub-Total						**$73,839,496**
Division 1 Cost		2.5%				$1,713,591
Engineering		16%				$11,814,319
Project Management/ Quality Control		4%				$2,953,580
Contingency		30%				$22,151,849
Sub-total						**$112,472,836**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$112,472,836**

LGWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 4, 2 X ADWF HRAS + PRIMARY

CAPITAL COSTS	# of module	Quantity per module	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	11427.792	14	159,989	m^3	$8	$1,279,913
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			6,500	m^2	$100	$650,000
Total for Site Improvement						$5,089,913
Treatment Components:						
Chemical Treatment	Allowance					$500,000
Primary Clarifiers	7.9	186	1,458	m^2	$4,056	$5,913,822
Fine Screens	-	-	0	ML/d	$21,413	$0
Trickling Filters	5.9	1382	8,184	m^3	$1,000	$8,184,375
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
Bioreactor	0.0	0	0	m^2		combined with TF
Secondary Clarifiers	4.0	908	3,668	m^2	$2,644	$9,699,783
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	3.6	113	403	m^2	$20,905	$8,430,436
Digesters	3.3	3110	10,372	m^3	$1,030	$10,678,656
Mechanical Dewatering (Centrifuge)	3.0			l.s.	$1,254,277	$3,762,831
SBR				m^3		$0
UV			157	ML/d PWWF	$10,000	$1,572,000
Odour Control	Allowance					$370,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1050 mm dia.)		1050	55.5	m	$1,050	$58,275
Piping (1200 mm dia.)		1200	26.5	m	$1,200	$31,800
Piping (1200 mm dia.)		1200	382	m	$1,200	$458,400
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			56,850,291	%	4.00%	$2,274,012
Electrical substation		1119	1	l.s.	$75,000	$75,000
Existing Facility Upgrades						$0
Sub-Total						**$59,199,303**
Division 1 Cost		2.5%				$1,352,735
Engineering		16%				$9,471,888
Project Management/ Quality Control		4%				$2,367,972
Contingency		30%				$17,759,791
Sub-total						**$90,151,689**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$90,151,689**

LGWWTP CAPITAL COST ESTIMATE: BUILD OUT TO SECONDARY OPTION 5, 2 X ADWF TF/SC IN PARALLEL WITH PRIMARY

CAPITAL COSTS	# of module	Quantity per module	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove sludge stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground densification - new	12360.264	14	173,044	m^3	$8	$1,384,350
Ground densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil anchors - existing			370	each	$4,000	$1,480,000
Dewatering			8,040	m^2	$100	$804,000
Total for Site Improvement						$5,348,350
Treatment Components:						
Chemical Dosing						$0
Primary Clarifiers	0.0	186	0	m^2	$4,056	$0
Fine Screens	-	-	262	ML/d	$21,413	$5,610,253
Trickling Filters	7.0	2608	18,375	m^3	$1,000	$18,375,000
Roughing Trickling Filters	0.0	0	0	m^3	$900	$0
Bioreactor	0.0	0	0	m^2	-	combined with TF
Secondary Clarifiers	6.7	908	6,113	m^2	$2,644	$16,166,306
Gravity Thickeners	0.0	147	0	m^2	$4,500	$0
DAF Thickeners	4.6	113	516	m^2	$20,905	$10,789,938
Digesters	1.4	3110	4,398	m^3	$1,030	$4,527,934
Mechanical Dewatering (Centrifuge)	1.0			l.s.	$1,254,277	$1,254,277
SBR				m^3		$0
UV			262	ML/d PWWF	$10,000	$2,620,000
Odour Control	Allowance					$845,000
Site Works:						
Pumping to Bioreactor			0	kW	$0	$0
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1600 mm dia.)		1600	300.5	m	$1,600	$480,800
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			68,117,857	%	4.00%	$2,724,714
Electrical substation		1348	1	l.s.	$85,000	$85,000
Existing Facility Upgrades						$0
Sub-Total						**$70,927,571**
Division 1 Cost		2.5%				$1,639,481
Engineering		16%				$11,348,411
Project Management/ Quality Control		4%				$2,837,103
Contingency		30%				$21,278,271
Sub-total						**$108,030,837**
Net GST (0%)		0%				$0
Total (Capital Costs)						**$108,030,837**

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 5
Pilot Scale Testing Program

FINAL REPORT

Prepared for

Greater Vancouver Regional District

Prepared by

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August 2005

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1 INTRODUCTION AND PROJECT OBJECTIVES

This Appendix contains the results of the small-scale treatability testing program that forms a component of the GVRD Facilities Plans for the Iona Island and Lions Gate WWTPs.

The small-scale testing program was designed to conduct parallel comparisons of the effectiveness of chemically enhanced primary treatment versus partial biological treatment in removing toxicity from the effluent at the Iona Island and Lions Gate WWTPs. The comparisons were undertaken using batch tests onsite at the plants.

The overall objective was to assess potential treatment schemes that can significantly improve the results of effluent toxicity testing at the Iona and Lions Gate WWTPs in the interim period leading up to the implementation of full secondary treatment at both plants, as set out in the GVRD Liquid Waste Management Plan. Significant improvement in toxicity reduction was measured by an increase in the frequency of success in passing the 96 hour LC_{50} acute toxicity bioassay (EPS 1/RM/13).

The proposed program for small-scale testing was submitted to the GVRD for review and comment by Memorandum on July 24, 2003. After discussion at a project meeting held at the GVRD on July 30, 2003, the draft program was extended to include evaluation of chemically-enhanced primary treatment (CEP) followed by partial biological treatment at the GVRD's request. The revised pilot-scale program was re-submitted to the GVRD for review by Memorandum on August 1, 2003. No further comments or requests were received, and pilot testing of the various treatment processes commenced on August 5, 2003. The ***Guideline for the release of ammonia dissolved in water found in wastewater effluents (The Federal Guideline)*** published on December 4, 2004 in terms of the Canadian environmental Protection Act,1999 requires the use of Reference Method EPS 1/RM/13 for determining the acute lethality of effluents. This test method was used in this project for the toxicity testing with the prior agreement of the GVRD.

GVRD staff have carried out a large number of toxicity investigations, in-house and through contracted research work, to identify the causes of toxicity at both Lions Gate and Iona Island WWTPs. The Stantec/Dayton & Knight small-scale treatability testing work used results of this work as the starting point for designing the treatability studies to compliment the GVRD investigation. The main objective of the small scale testing was to conduct treatability studies to reduce toxicity and not to conduct studies to identify causes of toxicity. The use of bench scale was proposed since the literature contains very little information on reducing toxicity using chemical or partial biological treatment.

The treatability testing was also performed on samples of primary effluent, which were collected at the time of day when the highest organic strength of wastewater was being experienced at Iona – around 12:00 noon to 2:00 pm. The high strength sampling is the most appropriate for treatability testing since one of GVRD's goals in plant design was assumed to be reduction in toxicity. The concern in this study was that the control samples would not display toxicity if they were taken in the morning when the wastewater is historically weaker.

2 BACKGROUND

Previous study has identified anionic surfactants, which are collectively measured as methylene blue active substances (MBAS), as the primary cause of toxicity at the Lions Gate WWTP. It has been reported that effluent MBAS concentrations in excess of about 2 mg/L to 2.5 mg/L correlate with increased failure of the 96-hour LC_{50} acute toxicity fish bioassay at Lions Gate (CH2M Hill 2002). Limited sampling and analysis previously conducted by others indicates that the influent MBAS concentration at Lions Gate is typically about 2-4 mg/L from 8 AM until late morning, and then increases to a peak as high as 10 mg/L to 11 mg/L by about 4 PM (CH2M Hill 2002; EVS 2001 and GVRD 2003). A single diurnal MBAS profile taken at Lions Gate on March 26 to 28, 2001 showed that the high MBAS concentration (8 mg/L to 10 mg/L) in the plant effluent lasted until about midnight (GVRD 2003).

At the Iona WWTP, low dissolved oxygen has been tentatively identified as the main cause of failures of the standard LC_{50} test. The low dissolved oxygen has been attributed to high oxygen demand in the plant effluent samples caused by an active population of viable microbes present in the plant influent, combined with high concentrations of readily-degradable organic material (BOD) in the primary treated effluent. A diurnal profile taken at Iona on August 12, 1996 showed that the plant influent BOD concentration was low during the early to late morning, and then increased steeply during the late morning to early afternoon, peaked during the later afternoon to early evening, and then declined during the early morning (CG&S 1996).

Limited pilot scale testing at the Iona WWTP has shown that CEP can meet the interim effluent requirements of 130 mg/L for BOD and 100 mg/L TSS for typical domestic wastewater at a chemical dose of about 75 mg/L ferric chloride and 1 mg/L anionic polymer, which reportedly improved removal of BOD and TSS by 35% to 60% and 65% to 95%, respectively (AE 1999a and 1999b). Others have recommended lower chemical doses of 10 mg/L to 30 mg/L ferric chloride and 0.1 mg/L to 0.3 mg/L polymer based on bench-scale and pilot- scale testing at Iona; this level of chemical addition improved removal of BOD and TSS by 7% to 10% and 15% to 25%, respectively (CG&S 1997/98). The CEP system currently in place at Iona, which is designed to be used on an intermittent basis if needed, is based on a chemical dose of 70 mg/L alum with 0.5 mg/L anionic polymer (Taw 2003).

Bench-scale testing at the Lions Gate WWTP showed that both ferric chloride and alum at a dose of 75 mg/L produced BOD and TSS removal efficiencies of 80% and 85%, respectively (effluent concentrations were 40 mg/L BOD and 30 mg/L TSS); the addition of anionic polymer at a dose of 0.25 mg/L significantly increased floc size and improved the settling rate (AE 1988). Stress testing of the primary tanks at Lions Gate in 1996 (no chemical addition) showed that TSS removal decreased from 80% at a surface overflow rate (SOR) of 70 $m^3/m^2/d$ to 50% at an SOR of 180 $m^3/m^2/d$, but was still within the effluent limit of 130 mg/L. It was estimated that chemical addition could reduce effluent BOD by about 30 mg/L during dry weather flows (CG&S 1996).

Bench-scale testing at Lions Gate resulted in about 50% removal of surfactants measured as methylene blue active substances (MBAS) using a dose of 30 mg/L to 50 mg/L alum, which was found to be more effective than ferric chloride (CH2M Hill 2002).

3 METHODOLOGY

The pilot-testing program was designed to conduct parallel tests on samples of settled sewage leaving the primary settling tanks. The purpose of the parallel tests was to compare the effectiveness of chemically enhanced primary treatment (CEP) with that of partial biological treatment, and also with that of CEP followed by partial biological treatment, in reducing the acute toxicity of the effluent at Lions Gate and Iona (acute toxicity as measured by the 96-hour LC_{50} rainbow trout bioassay). At Iona only, an additional batch test was included, to assess the effectiveness of chlorination/dechlorination in improving the chance of passing the 96 hour LC_{50}, by reducing the population of viable bacteria in the plant effluent sample and consequently reducing the oxygen demand during the bioassay.

For partial biological treatment, it was determined that acclimating and maintaining pilot-scale biological systems at the WWTPs over a period of 10-12 weeks was not practical, due mainly to schedule and budget limitations. Therefore, evaluation of partial biological treatment at both plants was undertaken using biological waste sludge taken from the Annacis Island WWTP. Development of an activated sludge specifically acclimatized to the Iona Island and Lions Gate wastes would have been a better approach. However, the organic components of the wastewater degraded satisfactorily in the batch test and provided confidence that the test results were typical of how effective biological treatment would be.

Each batch test was done in parallel onsite at either Lions Gate or Iona, using settled sewage from that facility, combined with waste biological sludge from Annacis. Review of grab sampling data supplied by the GVRD for the period 2001 to the present showed that the typical influent MBAS concentrations in the morning (around 8 AM to noon) at Lions Gate were usually in the range 2-4 mg/L, compared to MBAS concentrations of 2-5 mg/L in the Annacis influent over the same period. Effluent MBAS concentrations at Annacis were typically 0.2-0.3 mg/L (GVRD 2003). Therefore, it is apparent that the biomass at the Annacis WWTP is well acclimated to MBAS removal.

3.1 SCHEDULE

Testing was designed to incorporate the daily peak concentrations of the parameters of interest as far as possible, while maintaining reasonable working hours. The samples of settled sewage for use in the batch tests were taken during the day at Iona Island, since work by others showed that high influent BOD concentrations occurred during early afternoon to late evening (CG&S 1996). The diurnal MBAS profiles obtained during the current study (see Section 3.2 below) as well as previous work by others showed that the high influent MBAS concentration occurred around midnight at Lions Gate; the samples of settled sewage were accordingly obtained between midnight and 1 AM on each test day at Lions Gate, using a pump with a programmable timer. Test days at each plant were arranged to randomly cover as many days of the week as possible (Mon. Tues., Wed. etc.). Testing was carried out during the period August 5, 2003 through September 23, 2003. Virtually no precipitation occurred during the testing period.

3.2 DIURNAL MBAS PROFILES AT LIONS GATE AND IONA WWTP'S

To confirm the time of day for peak MBAS concentration in the influent sewage at Lions Gate, a 48-hour profile was obtained on July 21-23, 2003 using an automated sampler. The automated sampler obtained 500 mL samples at 1-hour intervals (24 samples in each 24-hour period). To minimize analysis costs, composite samples were then prepared representing each 3-hour period (i.e., eight 3-hour composite samples in each 24-hour period). The samples were analyzed for MBAS only. It was subsequently decided to obtain a similar 48-hour MBAS profile at the Iona Island facility, to compare the MBAS loading pattern between the two plants. The MBAS profile at Iona was obtained on September 10-12, 2003.

3.3 JAR TEST TO DETERMINE CHEMICAL DOSE FOR MBAS REMOVAL AT LIONS GATE WWTP

To confirm the optimum alum dose for MBAS removal at the Lions Gate WWTP, a bench-scale jar test was conducted on August 6, 2002. A sample of settled sewage was obtained at Lions Gate between midnight and 12:40 AM using the automated sampling pump. This sample time was selected on the basis of the diurnal MBAS profiles (see Section 3.2 above). The sample of settled sewage was then used in a jar test. Six samples plus one control were tested. Each sample contained 2 L of settled sewage. The control sample received no chemical addition. The six test samples received alum doses of 20 mg/L, 40 mg/L, 60 mg/L, 80 mg/L 100 mg/L, and 120 mg/L; each test sample also received 0.5 mg/L of anionic polymer (NALCO 1C34). All samples were mixed vigorously for 3 minutes using a standard rotary paddle-mix jar test device, followed by 30 minutes of gentle mixing and 1 hour of quiescent settling. Samples for MBAS analysis were then decanted from the top of each jar.

3.4 BATCH TEST PROCEDURE

a) Iona and LG - fill 400 L tank with settled sewage and mix well – remove 20 L sample for bioassay testing as well as samples for total suspended solids (TSS), total and soluble five-day biochemical oxygen demand (TBOD and SBOD), ammonia, and MBAS analysis – also remove enough settled sewage from 400 L tank for use in the other tests and to prepare sample mixtures for partial biological treatment (see Items c, d, and e below).

b) CEP at Iona and LG - add 70 mg/L alum and 0.5 mg/L polymer (NALCO 1C34 as used at Iona) to remaining settled sewage in 400 L tank, aerate/mix vigorously for 5 minutes followed by 30 minutes of gentle aeration/mixing, then shut off air and settle for 2 hours – decant 20 L sample for bioassay testing and samples for TSS, TBOD, SBOD, ammonia, and MBAS analysis.

c) Biological Treatment at Iona and LG – obtain 40 L sample of waste biological sludge from Annacis WWTP – add to 100 L tank 80 L settled sewage from 400 L tank and 20 L biological sludge - mix/aerate for 3 hours (monitor dissolved oxygen concentration using portable probe and try to maintain 2 mg/L), then shut off air and settle for 1 hour (note target F/M ratio was about 0.4/d) – obtain samples for TSS, TBOD, SBOD, ammonia, and MBAS analysis for three out of the six tests at each plant, to allow evaluation of contaminant removal by full (100%) biological treatment.

d) Samples for 50% Biological Treatment at Iona and LG - decant 10 L sample from biological batch reactor (Item c) and mix with 10 L settled sewage obtained previously from 400 L tank (see Item a) – send 20 L mixture for bioassay testing and prepare smaller samples in the same 50/50 proportion for TSS, TBOD, SBOD, ammonia, and MBAS analysis.

e) Samples for 25% Biological Treatment at Iona and LG - decant 5 L sample from biological batch reactor (Item c) and mix with 15 L settled sewage obtained previously from 400 L tank (see Item a) – send 20 L mixture for bioassay testing and prepare smaller samples in the same 25/75 proportion for TSS, TBOD, SBOD, ammonia, and MBAS analysis.

f) Samples for CEP Plus 25% Biological Treatment at Iona and LG- use remaining 20 L sample of waste biological sludge from Annacis WWTP (see Item c) – add to 100 L tank 80 L CEP treated sewage from Item b and 20 L biological sludge – then follow biological treatment procedure as described in Items c and e above.

g) Chlorination/Dechlorination at Iona only - take 20 L sample of settled sewage from 400 L tank before CEP is initiated (see Item a) – add sodium hypochlorite (high-strength industrial bleach) as needed to maintain chorine residual of at least 2 mg/L for 1 hour, mixing well after each addition – then dechlorinate using sodium thiosulfate and send 20 L sample for bioassay testing – note that this test was dropped after three replicates, since all three of the treated samples failed to pass the toxicity bioassay (see Section 4.3 Results).

3.5 SAMPLE HANDLING AND ANALYSIS

All samples for bioassay and chemical testing were put on ice immediately after being obtained. Where possible, the samples were transported to the laboratory immediately after the batch test was completed. Due to the sampling schedule, the samples from the Iona WWTP for Tests 4, 5 and 6 had to be kept on ice overnight and delivered to the laboratory the next morning.

a) Bioassays (EVS laboratory) – on each testing day, 1 sample at each plant for settled sewage, 1 for CEP effluent, 1 for 25% biological treatment, 1 for 50% biological treatment and 1 for CEP + 25% biological treatment plus 1 sample at Iona for chlorination/dechlorination – all bioassay tests were conducted according to the Environment Canada protocol for acute toxicity using rainbow trout (EPS 1/RM/13, Second Edition, 2000) – an independent EVS QA/QC review confirmed that all acceptability criteria specified by the protocols were met.

b) Chemical Testing (Cantest laboratory) – same samples as described in Item a above for bioassays were obtained for chemical testing (TBOD, SBOD, TSS, ammonia, MBAS) – also samples were obtained for chemical testing (TBOD, SBOD, TSS, ammonia, MBAS) from the undiluted (100%) biologically treated sewage for three randomly selected tests at each plant.

c) Materials and Equipment

- four 100 L tanks and two 400 L tanks
- 1 portable dissolved oxygen meter
- 1 portable pH/temperature meter
- 1 chlorine residual kit
- air tubing, aeration diffusers, valves
- alum and polymer (NALCO 1C34)
- sodium hypochlorite for chlorination
- sodium thiosulfate for dechlorination

4 RESULTS AND DISCUSSIONS

4.1 DIURNAL MBAS PROFILES AT LIONS GATE AND IONA WWTP'S

The historical results of the diurnal MBAS profiles conducted by others on the plant influent at the Lions Gate WWTP are summarized on Figure 4.1. The results of the MBAS profiles obtained during the current study on the settled sewage leaving the primary tanks at the Lions Gate plant are shown on Figure 4.2 for comparison. As shown, the data collected during this study for Lions Gate (Figure 4.2) are consistent with the historical data (Figure 4.1). The MBAS concentration at Lions Gate was typically low (2 mg/L to 4 mg/L) from mid-morning to early afternoon (8 A.M. to about 3 P.M), and then increased by around 4 P.M to about 6 mg/L. The MBAS concentration continued to increase until it peaked at 8 mg/L to 10 mg/L around midnight, and then began to decline during the early morning hours (1 A.M. to 7 A.M.). Based on the data obtained, it was decided to obtain the samples of settled sewage for batch testing at Lions Gate between the hours of midnight and 1 A.M., to test the effectiveness of the various treatment schemes under times of maximum MBAS concentration.

FIGURE 4.1
LIONS GATE WWTP HISTORIC DIURNAL MBAS PROFILE

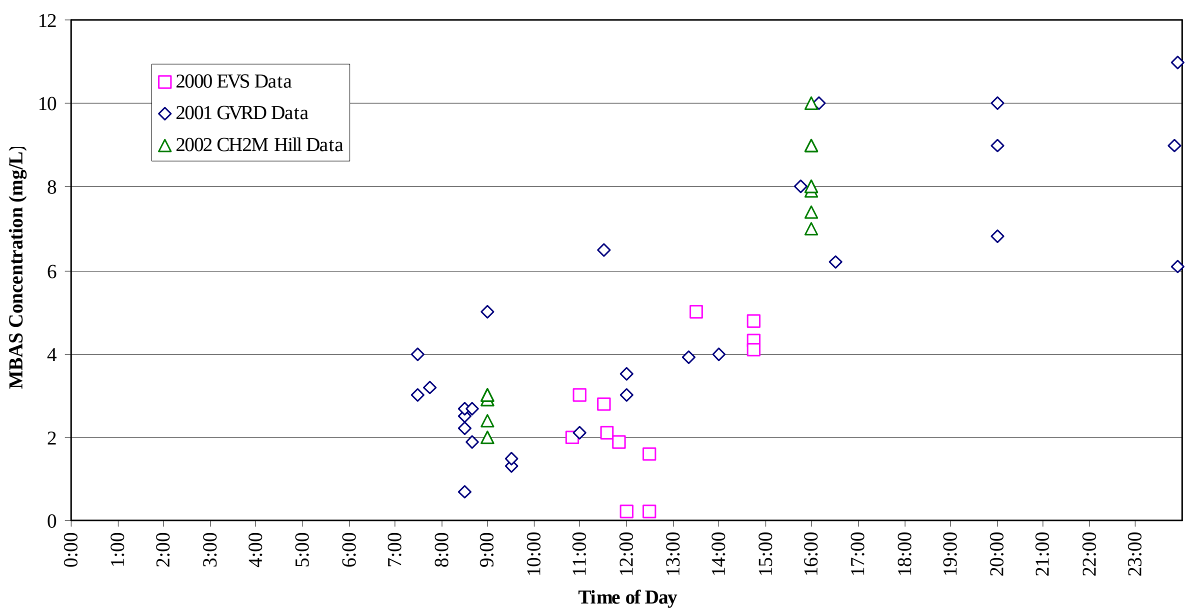
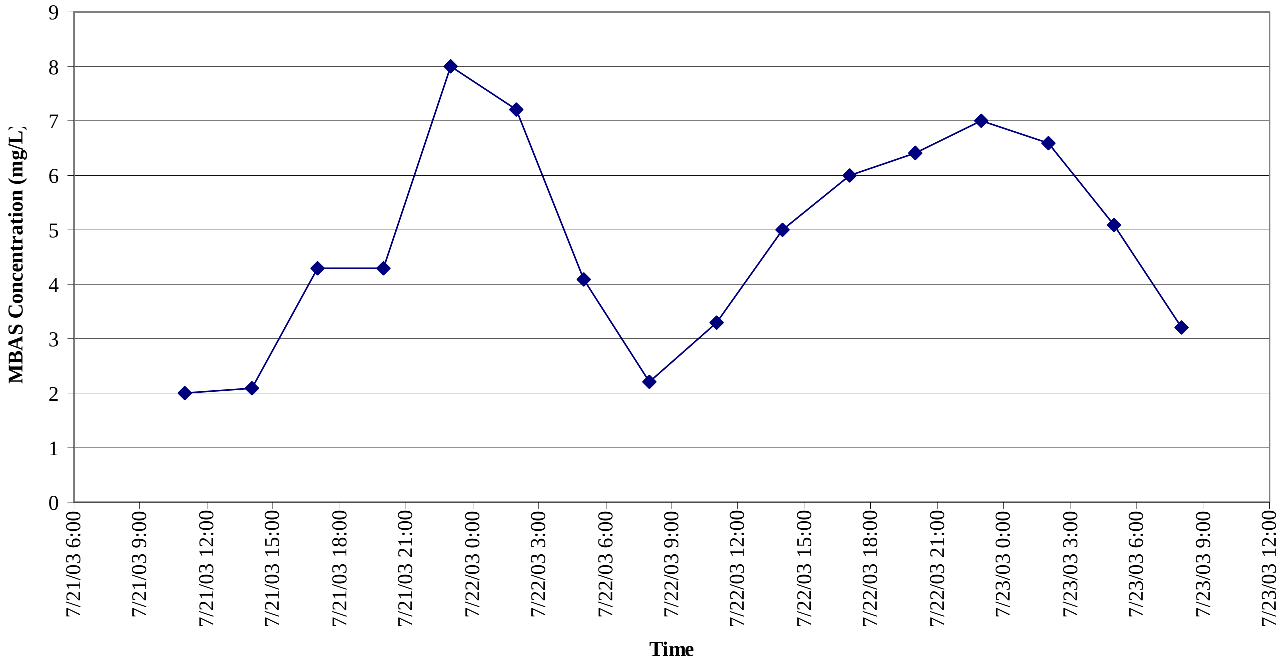
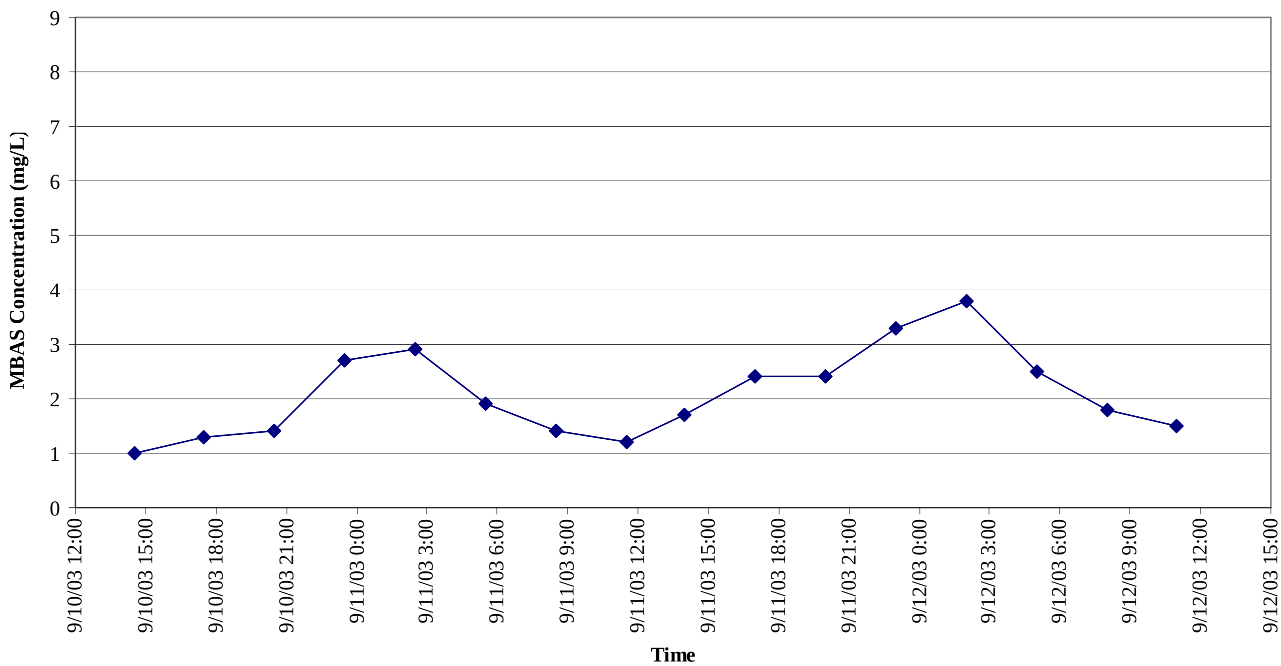


FIGURE 4.2
LIONS GATE WWTP HISTORIC DIURNAL MBAS PROFILE, JULY 21-23, 2003



The results of the diurnal MBAS profiles conducted on the primary effluent at the Iona Island WWTP are shown on Figure 4.3. The diurnal pattern observed in this study was similar to that observed at Lions Gate, except that the peak MBAS concentration at Iona occurred slightly later (around 3 AM compared to midnight at Lions Gate), and the peak MBAS concentration at Iona was much lower (3 mg/L to 4 mg/L compared to 7 mg/L to 8 mg/L at Lions Gate). Since oxygen demand rather than MBAS was identified as the primary toxicity issue at Iona Island affecting the LC_{50} test performance, the sampling schedule at Iona was not adjusted to obtain samples of settled sewage for batch testing during times of peak MBAS concentration.

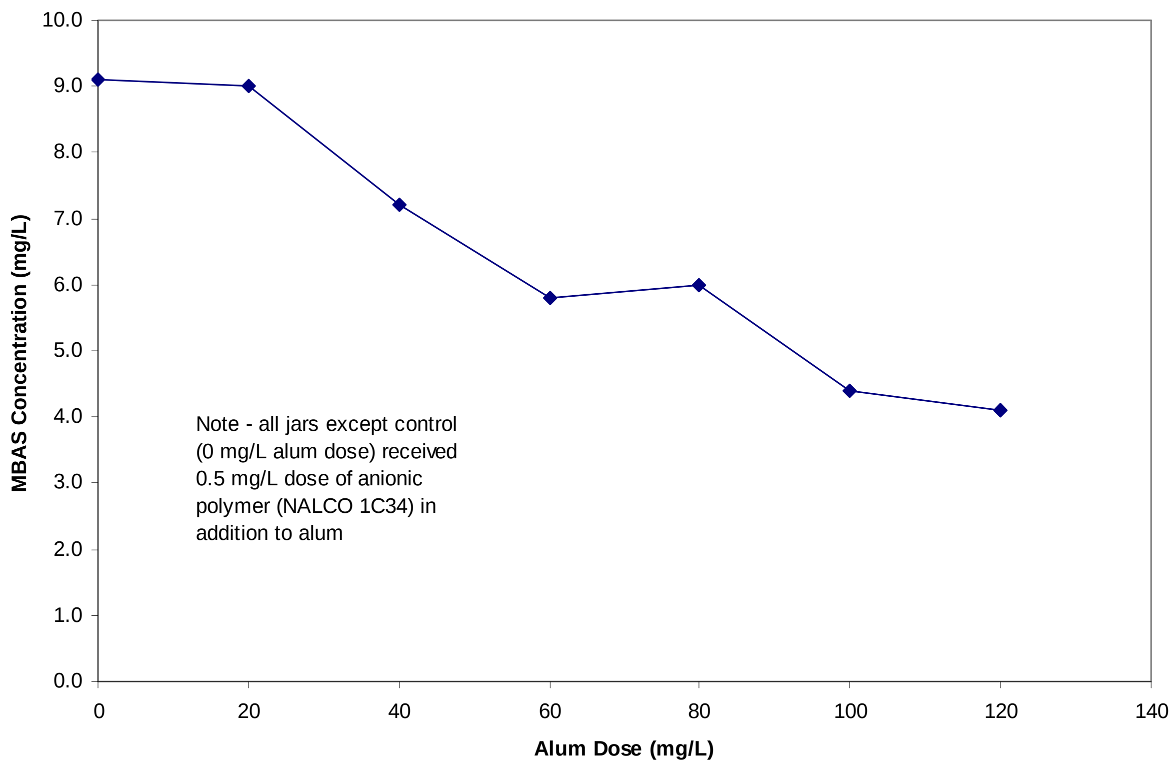
FIGURE 4.3
IONA ISLAND WWTP DIURNAL MBAS PROFILE SEPT. 10-12, 2003



4.2 JAR TEST TO DETERMINE CHEMICAL DOSE FOR MBAS REMOVAL - LIONS GATE WWTP

The results of jar testing to evaluate the optimum chemical dose for MBAS removal are summarized on Figure 4.4. As shown, a dose of 20 mg/L alum resulted in little improvement compared to the control (zero alum addition). A relatively steep improvement in MBAS removal was observed as the alum dose increased from 20 mg/L to 60 mg/L. Additional improvement began to decline as the alum dose was increased to 120 mg/L. The maximum achievable MBAS removal under the conditions of this test appeared to be about 55% removal (from about 9 mg/L to about 4 mg/L). Based on the results of this test and economic analyses prepared by others, it was determined to use a chemical dose of 70 mg/L alum with 0.5 mg/L anionic polymer (as currently practiced on an intermittent basis at Iona) for the CEP batch tests at Lions Gate WWTP. Further testing at full-scale will be needed to confirm the optimum chemical dose for MBAS removal at Lions Gate.

FIGURE 4.4
LIONS GATE WWTP JAR TEST FOR MBAS REMOVAL



4.3 BATCH TEST AT IONA ISLAND WWTP

It is important to emphasize that the results discussed below were based on only six grab samples taken at each of the two WWTPs over a 6 to 8 week dry weather period. Removal of each chemical parameter has been expressed as an average over the six tests at each plant, to simplify discussion of the results. However, as shown by the detailed results in the appendices, there was wide variation in the results among the test replicates (standard deviations were in some cases larger than the means). Comparisons among the various treatments should be taken as subjective; that is, since parallel tests were conducted on the same sample of settled sewage each time, relative comparisons regarding the effectiveness of one treatment compared to the others are valid. However, the average percentage removals of chemical parameters and the percentage improvement in the frequency of toxicity associated with the various treatment processes in the batch tests should not be projected to full-scale WWTP performance. Our experience with full-scale operation of flow through activated sludge treatment plants is that they usually show better performance than bench scale batch tests.

It is also important to note that this study was aimed not only at providing information on the effectiveness of various treatment approaches in improving the results of the 96-hour LC_{50} acute toxicity bioassay for effluent testing at the Iona Island and Lions Gate WWTPs but also to provide information on improvement to compliance parameters such as BOD and TSS. Detailed Toxicity Identification and Evaluation (TIE) studies were not included in the scope of work for this project. Because of the strict protocols that are applicable to Reference Method EPS 1/RM/13, pH conditions can change during the test generating concentrations of un-ionized ammonia that can increase the mortality rates of the test organisms from the level pertinent to the original wastewater sample.

It should be noted in reviewing the batch test results that for the first test at Iona, the polymer dose was 200 mg/L rather than 0.5 mg/L as specified in the Methodology (this was due to incorrect information received regarding polymer application at the full scale plant).

The detailed results of chemical testing for each individual test at Iona are contained in Appendix A. A summary comparison of the results for both Iona and Lions Gate is contained in Appendix C.

The results of the acute toxicity bioassay testing at Iona (96 hr LC_{50}) are summarized In Table 4.1 (see Appendix A for more detail). As shown, the control sample (untreated settled sewage) was acutely toxic for Tests 1 to 5, and was non-acutely toxic for Test 6. The results of Test 6 were accordingly not included in the evaluation of toxicity reduction. Three of the treatment processes (CEP, 50% biological, and CEP+25% biological) produced three non-acutely toxic samples and two acutely toxic samples over Tests 1 to 5. Twenty-five percent biological treatment produced only one non-acutely toxic sample over Tests 1 to 5. All three samples produced by disinfection (Tests 1 to 3) were acutely toxic.

The “acute lethal concentration of ammonia” as defined in The Federal Guideline results in more than a 50% kill of rainbow trout over 96 hours. The ammonia concentrations associated with this have been calculated using the formula in The Federal Guideline for the pH values recorded in the tests at t=0 hrs and t=24 hours and are set out in Appendix A. The resulting acute lethal concentrations of ammonia, in all but one case, are higher (factor 1.5 to 7.9) than the concentrations measured in the tests. This indicates that ammonia toxicity is most unlikely to be the primary cause of failure of the tests.

The concentration of un-ionized ammonia 24 hr after the start of the bioassay tests is indicated in Table 4.2. The pH at that time was selected since fish kill during the test generally occurred during the first 24 hours of the test. .Based on data from Environment Canada, the mean lethal (LC_{50}) concentration for un-ionized ammonia for rainbow trout is 0.481 mg/L for the standard 96 hour bioassay. The mean lethal concentration of 0.481 mg/L reflects the actual concentration on un-ionized ammonia that causes the 96 hour bioassay to fail. As indicated in Table 4.2, the un-ionized concentration of ammonia during the test is well below the mean lethal concentration of 0.481 mg/L except for test # 5 at 50% biological which was only slightly below 0.481 mg/L. This indicates that ammonia toxicity is most unlikely to be the primary cause of failure of the tests.

TABLE 4.1
IONA ISLAND WWTP TOXICITY RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Pass
CEP	Fail	Pass	Pass	Pass	Fail	Pass
25% Biological	Pass	Fail	Fail	Fail	Fail	Pass
50% Biological	Pass	Pass	Pass	Fail	Fail	Pass
CEP+25% Biological	Pass	Pass	Pass	Fail	Fail	Pass
Disinfected	Fail	Fail	Fail	N/A	N/A	N/A

TABLE 4.2
CONCENTRATION OF UN-IONIZED AMMONIA @ 24 HR DURING TEST

Treatment	Test # 1	Test # 2	Test # 3	Test # 4	Test # 5	Test # 6
Control	0.125	0.107	0.225	0.128	no data	0.089
CEP	0.107	0.154	0.271	0.186	0.178	0.080
25% biological	0.318	0.211	0.209	0.186	0.182	0.083
50% biological	0.331	0.317	0.345	0.174	0.465	0.121
CEP + 25% bio.	0.234	0.220	0.290	0.188	0.202	0.084

The result of Test # 1 to # 5 are summarized in Table 4.3. As indicated above, the results of Test # 6 were not included in the evaluation since the control test passed the toxicity test.

TABLE 4.3
SUMMARY IONA ISLAND TOXICITY RESULTS

Treatment	Pass	Fail	Remark
Control	0	5	LC_{50} NH_3 no exceedance
CEP	3	2	LC_{50} NH_3 no exceedance
25% Biological	1	4	LC_{50} NH_3 no exceedance
50% biological	3	2	LC50 NH_3 no exceedance
CEP + 25% biological	3	2	LC_{50} NH_3 no exceedance

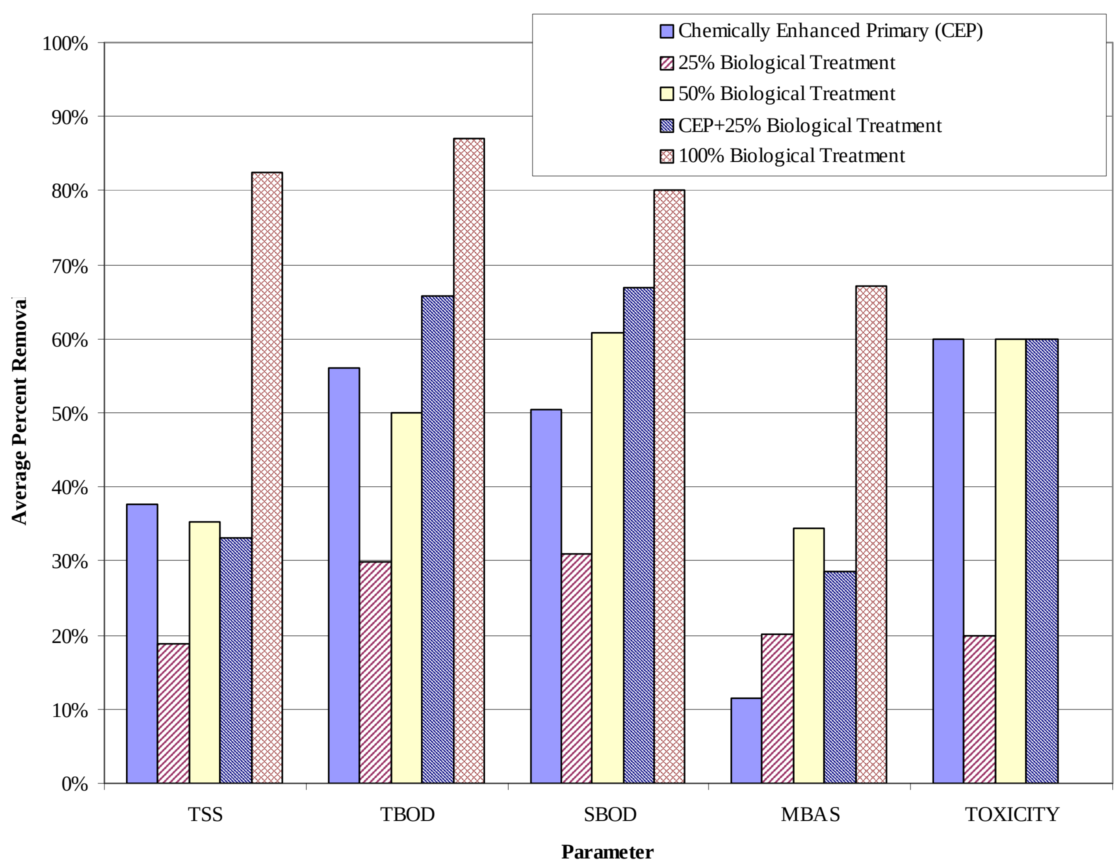
The data show, the toxicity observed was due to something other than ammonia. The un-ionized ammonia mean lethal concentration was not exceeded during the test as shown in Table 4.2.

The results of the chemical testing and the bioassay results for Tests 1 to 6 at Iona are summarized on Figure 4.5. The results shown on Figure 4.5 represent the average removal over the six tests for each parameter of interest (except for the toxicity results, which are based on Tests 1 to 5 only).

As shown on Figure 4.5, the performance of 25% biological treatment was substantially below that of all other treatments for all parameters except for MBAS (20% removal), where it was better than CEP at 12% MBAS removal but less than 50% biological treatment (34% MBAS removal) and CEP+25% biological treatment (29% MBAS removal).

Average removal of TSS was similar for CEP, 50% biological treatment, and CEP+25% biological treatment (i.e., 30% to 40% TSS removal). Average removal of TBOD and SBOD was in the range 60% to 70% for CEP+25% biological treatment, compared to 50% to 60% TBOD and SBOD removal for 50% biological treatment and CEP alone. From the standpoint of TSS and BOD removal, these three processes appear to be approximately equivalent, although CEP+25% biological treatment showed slightly better BOD removal than the other two processes.

FIGURE 4.5
IONA ISLAND WWTP SUMMARY OF BATCH TEST RESULTS



As indicated in Table 4.1, the bioassay results showed that CEP, 50% biological treatment, and CEP+25% biological treatment all produced a non-toxic effluent sample three out of five times; this represents a 60% improvement over the control, which was toxic five out of five times (neglecting the sixth test, where the control was non-toxic). The 25% biological treatment produced a non-toxic effluent sample only once out of five times, which represents a 20% improvement compared to the control.

As expected, 100% biological treatment resulted in substantially better removal of TSS, TBOD, SBOD, and MBAS than all other treatments (toxicity testing was not carried out for 100% biological treatment because of budget constraints). Removal of ammonia was less than 10% for all treatments and is not shown on Figure 4.5. Ammonia concentrations during the tests ranged from 12 to 25 mg/L, and was fairly consistent among the various treatments for each individual test - see Appendix A for details).

As shown in Appendix A, the control samples in Tests 1 to 5 at Iona showed low initial dissolved oxygen (DO) concentration at the outset of the bioassay test immediately following the pre-aeration period (i.e., less than 2 mg/L at t = 0 in Tests 1, 3, 4 and 5 and less than 3 mg/L in Test 2). Oxygen starvation was the most probable cause of the observed 100% mortality within the first hour in these five control samples. The sixth control sample, which passed the bioassay, had an initial DO concentration of 4.8 mg/L in the bioassay. It should be noted that 100% of fish death in control samples in Tests 1, 3, 4 and 5 occurred in less than one hour. This is an indication that the increased in non-ionized ammonia toxicity resulting in the rise in pH is not the primary cause of test failure.

Disinfection substantially reduced the initial oxygen demand compared to the control during the bioassay, but this did not improve the bioassay results (see Appendix A). The initial DO concentration at t = 0 in the bioassay test of the disinfected samples was in some cases the same or higher than the initial DO for other treatments that passed the bioassay. For example, in Tests 1 and 2 the disinfection samples (100% lethality) had higher initial DO than the 50% biological treatment samples (100% survival). This indicates that reducing the initial oxygen demand by disinfection under the conditions used in this study (i.e., maintaining a total chlorine residual of about 2 mg/L for one hour) will probably not improve toxicity testing results. More detailed studies conducted by the GVRD of the impact of chlorination at higher dosages of 4, 8 and 12 mg/L and for extended contact periods of one to four hours showed that survival of test fish was greater than 50% for most samples analyzed in comparison to 0% survival for untreated samples.

Chlorination to those levels also resulted in significantly higher DO levels in the bioassay test containers.

Stantec and Dayton & Knight have a concern that at these increased levels of chlorine dosage, for long retention times, chlorinated organic compounds could develop in concentrations that could become problematic.

The results discussed above are consistent with the observations of soluble biochemical oxygen demand (SBOD), which was in the range 49 mg/L to 78 mg/L for the control in Tests 1 to 5, but was only 33 mg/L for the control in Test 6 (see Appendix A). The SBOD was usually reduced to 35 mg/L or less by the various batch treatment processes;

this did not always result in an improved end result in the bioassay test, but in one case it did extend the time required for at least 50% mortality to occur (see Test 5 results in Appendix A).

As shown by the bioassay results in Appendix A, there was a general tendency for the sample pH to increase over the course of the bioassay test, a phenomenon which is attributed to gas stripping of carbon dioxide due to aeration, and which increases the toxicity of ammonia. The ammonia concentration of each test sample is included in Appendix A (this is the ammonia concentration of the samples sent for chemical analysis at the conclusion of the batch test, and this can be assumed to be the ammonia concentration of the sample as received for bioassay testing). The pH of the samples in the bioassay test at t=0 and at t=24 hours are included in Appendix A, together with the acute lethal ammonia concentration calculated in accordance with the equation given in The Federal Guideline at the recorded pH of the sample during the bioassay at t=0 and t=24 hours.

In summary, the batch test results confirmed that oxygen demand is the primary cause of fish mortality in the 96-hour LC_{50} acute toxicity bioassay at the Iona WWTP. Three of the treatment processes tested, namely CEP, 50% biological, and CEP+25% biological, produced a significant improvement in effluent toxicity testing, although none of the three produced a sample that was consistently non-toxic. Disinfection using a total chlorine residual of 2 mg/L and a contact time of one hour reduced initial oxygen demand in the 96-hour LC_{50}, but did not improve the end result in the three tests.

4.4 BATCH TEST AT LIONS GATE WWTP

The limitations discussed at the beginning of Section 4.3 for the Iona WWTP also apply to the results discussed in this section for Lions Gate.

The detailed results of chemical and bioassay testing for each individual test are contained in Appendix B. A summary comparison of the results for both Iona and Lions Gate is contained in Appendix C.

The results of the acute toxicity bioassay testing at Lions Gate (96 hr LC_{50}) are summarized In Table 4.2 (see Appendix B for more detail). As shown, the control sample (untreated settled sewage) was acutely toxic for all six tests. The CEP+25% biological treatment produced a non-toxic sample five out of six times, although mortalities occurred in three of the tests that were found to be non-toxic. Two of the treatments (CEP and 50% biological) produced two non-acutely toxic samples and four acutely toxic samples over Tests 1 to 6. Twenty-five percent biological treatment produced no non-acutely toxic samples over Tests 1 to 6.

The ammonia concentrations associated with “acute lethal concentration of ammonia” as defined in The Federal Guideline have been calculated using the formula in The Federal Guideline for the pH values recorded in the tests at t=0 hrs and t=24 hours and are set out in Appendix B. The resulting acute lethal concentrations of ammonia, in all but one case, are higher(factor 2.3 to 7.9) than the concentrations measured in the tests. The lowest factor 1.25 was associated with a test that passed. This indicates that ammonia toxicity is most unlikely to be the primary cause of failure of the tests.

The concentration of un-ionized ammonia 24 hr after the start of the bioassay tests at Lions Gate is indicated in Table 4.5. The pH at that time was selected since fish kill during the test occurred during the first 24 hours of the test except for four tests where fish death occurred at 48 or 72 hours. Based on data from Environment Canada, the mean lethal (LC_{50}) concentration for un-ionized ammonia for rainbow trout is 0.481 mg/L for the standard 96 hour bioassay. The mean lethal concentration of 0.481 mg/L reflects the actual concentration on un-ionized ammonia that causes the 96 hour bioassay to fail. As indicated in Table 4.5, the un-ionized concentration of ammonia during the test is well below the mean lethal concentration of 0.481 mg/L except for test # 2 at CEP plus 25% biological. This indicates that ammonia was not the primary cause of toxicity.

TABLE 4.4
LIONS GATE WWTP TOXICITY RESULTS

Treatment	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Control	Fail	Fail	Fail	Fail	Fail	Fail
CEP	Fail	Pass	Fail	Pass	Fail	Fail
25% Biological	Fail	Fail	Fail	Fail	Fail	Fail
50% Biological	Fail	Pass	Pass	Fail	Fail	Fail
CEP+25% Biological	Pass	Pass	Pass	Fail	Pass	Pass

TABLE 4.5
COMPARISON OF UN-IONIZED AMMONIA DURING TEST
WITH LC_{50} (LETHAL) CONCENTRATION @ 20^0C

Treatment	Test # 1	Test # 2	Test # 3	Test # 4	Test # 5	Test # 6
Control	0.129	0.252	0.125	no data	0.211	no data
CEP	0.126	0.307	0.261	0.108	0.168	0.064
25% biological	0.145	no data	0.235	no data	0.173	no data
50% biological	0.143	0.296	0.194	0.205	0.288	no data
CEP + 25% bio.	0.315	0.518*	0.201	0.151	0.289	0.168

* Exceeds mean lethal concentration of 0.481 mg/L

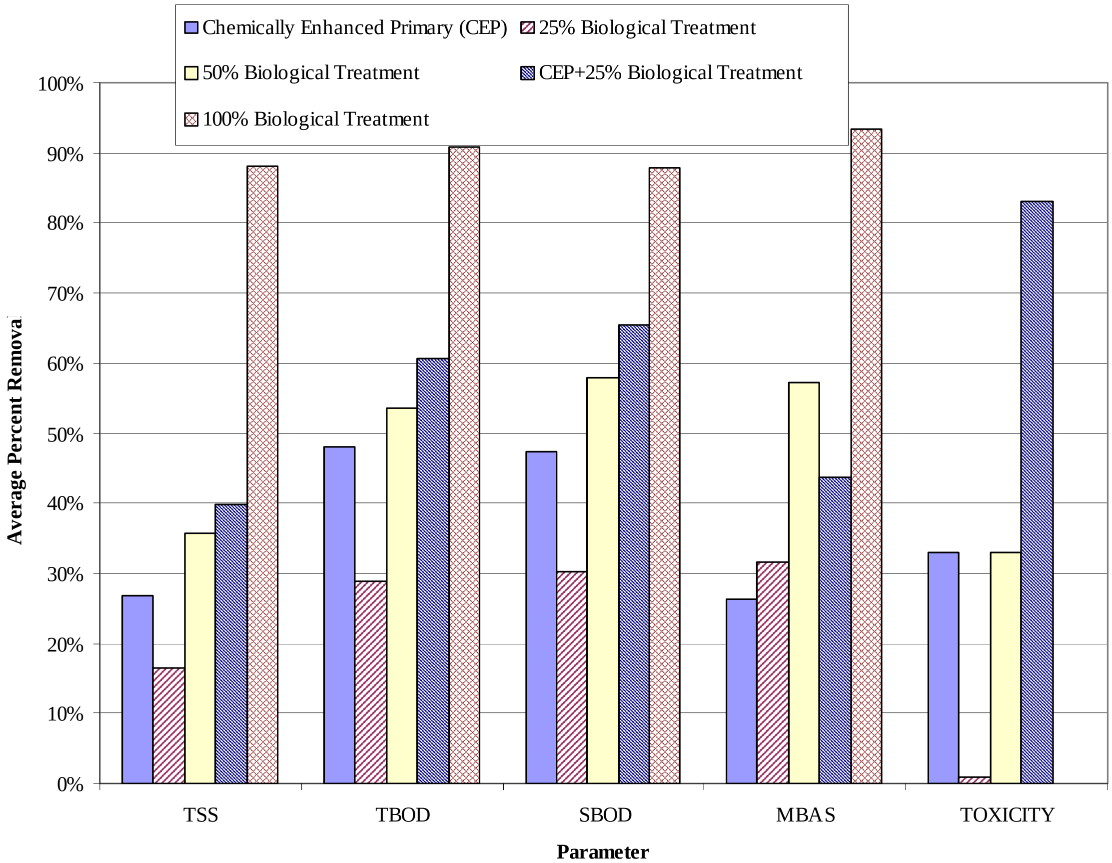
The results of Test # 1 to # 6 are summarized in Table 4.6. The data show that the toxicity observed was mainly due to something other than ammonia. The un-ionized ammonia mean concentration was only exceeded once as shown in Table 4.5.

TABLE 4.6
IONA ISLAND TOXICITY RESULTS FOR TESTS WITH NON TOXIC UN-IONINZED AMMONIA LEVEL

Treatment	**Pass**	**Fail**	**Remark**
Control	0	6	LC_{50} NH_3 no exceedance on 4 tests
CEP	2	4	LC_{50} NH_3 no exceedance
25% Biological	0	6	LC_{50} NH_3 no exceedance on 3 tests
50% biological	2	4	LC_{50} NH_3 no exceedance on 5 tests
CEP + 25% biological	5	1	LC_{50} NH_3 one exceedance

The results of the chemical testing and the bioassay results for Tests 1 to 6 at Lions Gate are summarized on Figure 4.6. The results shown on Figure 4.6 represent the average removal over the six tests for each parameter of interest.

FIGURE 4.6
LIONS GATE WWTP SUMMARY OF BATCH TEST RESULTS



The results for Lions Gate shown on Figure 4.6 exhibit a similar pattern to the results discussed earlier for Iona (Figure 4.5). As at Iona, the performance of 25% biological treatment at Lions Gate was substantially below that of all other treatments for all parameters except for MBAS. Removal of MBAS at Lions Gate for 25% biological treatment was 32%, compared to 26% for CEP. The 50% biological treatment at Lions Gate resulted in 57% MBAS removal, and CEP+25% biological treatment resulted in 44% MBAS removal. Percent removals of MBAS were generally higher at Lions Gate than at Iona, probably because of the higher initial MBAS concentration at Lions Gate.

Average removal of TSS was similar for 50% biological treatment (40% removal) and CEP+25% biological treatment (36% TSS removal), compared to 27% TSS removal for CEP. Average removal of TBOD and SBOD was highest for CEP+25% biological treatment at 60% to 65%, followed by 50% biological treatment at 50% to 60% BOD removal and CEP at slightly less than 50% BOD removal.

The bioassay results showed that CEP+25% biological treatment produced a non-toxic effluent sample five out of six times, which represents an 83% improvement over the control, which was toxic six out of six times. The CEP and 50% biological treatments produced a non-toxic effluent sample two out of six times, which represents an improvement of 33% compared to the control. The 25% biological treatment produced a toxic effluent sample six out of six times (i.e., no improvement compared to the control).

As at Iona, 100% biological treatment resulted in substantially better removal of TSS, TBOD, SBOD, and MBAS than all other treatments (toxicity testing was not carried out for 100% biological treatment). Removal of ammonia was less than 10% for all treatments and is not shown on Figure 4.6 (ammonia concentration was fairly consistent among the various treatments for each individual test - see Appendix B for details).

As shown in Appendix B, and similar to the samples discussed for Iona in Section 4.3, the control samples in Tests 1, 3, 4, and 6 at Lions Gate showed low initial dissolved oxygen (DO) concentration (less than 3 mg/L) at the outset of the bioassay test immediately following the pre-aeration period. Oxygen starvation was the most probable cause of the observed 100% lethality within the first 1-2 hours in these four samples. The control samples in Tests 2 and 5, which also caused 100% mortality but within a longer time frame (4 to 24 hrs), had initial DO concentrations of 4.4 mg/L and 5 mg/L, respectively. This shows that high oxygen demand was the primary cause of toxicity in four of the six tests conducted at Lions Gate, although additional (secondary) causes of toxicity may have also been present (see discussion of ammonia and MBAS toxicity below).

For the Iona results discussed in Section 4.3, low initial DO in the bioassay was consistent with the observations of soluble biochemical oxygen demand (SBOD) in the samples. However, this was not the case at Lions Gate, where the SBOD in Tests 2 and 5 (which had the highest initial DO) was not significantly lower than that in the other tests (Appendix B). The average concentration of SBOD in the six samples of primary effluent at Lions Gate was 83 mg/L, compared to only 55 mg/L at Iona.

As shown by the bioassay results in Appendix B and similar to the results discussed in Section 4.3 for Iona, there was a general tendency for the sample pH to increase over the course of the bioassay test at Lions Gate. Similar to the SBOD concentration discussed above, the ammonia concentration in the samples at Lions Gate was generally higher than that at Iona, indicating a less dilute sewage at Lions Gate (this despite the fact that the samples at Lions Gate were taken in the middle of the night and several of the samples at Iona were taken in the early afternoon to coincide with the maximum BOD concentration).

The pH of the samples in the bioassay tests for at Lions Gate at t=0 and at t=24 hours is included in Appendix B, together with the acute lethal ammonia concentration calculated in accordance with the equation given in The Federal Guideline at the recorded pH and temperature of the sample during the bioassay at t=0 and t=24 hours. As at Iona, the ammonia concentration of the control sample and the treated samples was similar among all of the samples within each batch test, regardless of the treatment process.

As described earlier in this Memorandum, the primary cause of acute toxicity in the effluent at Lions Gate has been tentatively identified by others as anionic surfactants (measured as methylene blue active substances or MBAS), and the toxicity threshold for MBAS has been estimated at 2 mg/L to 2.5 mg/L. However, the findings of this study did not show a correlation between MBAS concentration and acute toxicity, nor were they consistent with an MBAS toxicity threshold of 2.5 mg/L. As shown in Appendix B, 100% survival in the bioassay tests was observed in several treated samples that contained MBAS concentrations in the range 4 mg/L to 6 mg/L, and 90% survival was observed in one sample that contained 7.8 mg/L MBAS (see CEP for Test 4). Further, in three of the six batch tests, samples that were found to be non-acutely toxic according to the bioassay had higher MBAS concentrations than other samples within the same batch test that were found to be acutely toxic. The concentration of MBAS is therefore not a reliable indicator of fish toxicity, nor does the greatest degree of MBAS removal result in the greatest improvement in toxicity testing results.

The analysis for MBAS encompasses a large number of individual compounds, which may have varying degrees of toxicity, and which may be removed in different amounts by different treatment processes. This could account for the apparent inconsistencies in the MBAS results discussed above. Further study is needed to identify individual MBAS compounds, their toxicity, and the effectiveness of biological versus chemical treatment in reducing the most toxic compounds to non-toxic levels. These issues are currently being investigated jointly by the University of British Columbia Department of Civil Engineering and by GVRD.

The inability to identify a single cause of mortality could result from the combined effects of a number of different concentrations of pollutants and reduced oxygen concentration.

5 CONCLUSIONS

The following conclusions are based on the results of this study (the conclusions should be considered in light of the limitations described at the beginning of Section 4.3 of this Memorandum).

5.1 GENERAL

1. The primary effluent from the Iona Island WWTP was acutely toxic five out of six times and the effluent from the Lions Gate WWTP was acutely toxic six out of six times during this study. It should be noted that routine sampling by the GVRD for effluent toxicity testing normally occurs at around 8 to 9 A.M. at both plants, and that the effluent sampling times used in this study were around midday at Iona Island and shortly after midnight at Lions Gate.

2. Twenty five percent biological treatment was relatively ineffective in improving removal of TSS, TBOD, and SBOD at both Iona and Lions Gate compared to the other treatment processes, and was similarly ineffective in reducing the frequency of acute toxicity in the effluent at both plants. Twenty five percent biological treatment appears to hold little promise for achieving significant interim improvements at either Iona Island or Lions Gate.

3. The influent wastewater during dry weather at the Iona Island WWTP is more dilute than the influent at the Lions Gate WWTP.

4. Ammonia toxicity is most unlikely to be the primary cause of the failure of the tests because the concentration of un-ionized ammonia during the test is generally well below the mean lethal concentration.

5.2 IONA ISLAND WWTP

1. The samples of primary effluent from the Iona Island WWTP contained material that exerted a high oxygen demand in five of the six batch tests. Oxygen starvation was the most probable cause of the observed 100% lethality within the first hour in the control samples in these five tests. This conclusion is consistent with the findings of others reviewed in Section 2 of this Appendix.

2. Disinfection of the primary effluent at Iona Island was effective in reducing the initial oxygen demand in the bioassay test, but this did not improve the bioassay results. This indicates that reducing the initial oxygen demand by disinfection under the conditions used in this study (i.e., maintaining a total chlorine residual of 2 mg/L for a contact time of one hour) will not improve toxicity testing results.

3. The concentration of anionic surfactants (measured as methylene blue active substances or MBAS) in the effluent at the Iona Island WWTP appears to follow a similar diurnal cycle to that at the Lions Gate WWTP (see Item 3 below).

However, the concentration of MBAS at Iona Island is much lower than at Lions Gate.

4. Three of the processes tested at Iona were equal in terms of reducing the frequency of toxicity, these were chemically enhanced primary (CEP), 50% biological treatment, and CEP followed by 25% biological treatment. All three processes showed a 60% improvement in toxicity testing results compared to the control.

5. Chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment all showed similar removals of TSS. Chemically enhanced primary treatment followed by 25% biological treatment was slightly better for BOD removal than 50% biological or CEP alone.

6. Chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment appear to be approximately equivalent for use as interim improvements at Iona from the standpoint of toxicity reduction and removal of TSS and BOD. None of these processes will produce an effluent that is consistently non-toxic according to the 96 hour LC_{50}, but all can be expected to effect substantial improvements over primary treatment alone.

5.3 LIONS GATE WWTP

1. The samples of primary effluent from the Lions Gate WWTP contained material that exerted a high oxygen demand in four of the six batch tests. For these four control tests, 100% lethality of test fish was observed within the first two hours. Oxygen starvation, in conjunction with MBAS concentration, could have contributed to the observed toxicity. This conclusion is not necessarily inconsistent with previous work by others reviewed in Section 2 of this Memorandum, which identified anionic surfactants (measured as methylene blue active substances or MBAS) as the primary cause of toxicity at Lions Gate. The differences in wastewater characteristics at the later in the day sampling time could have accounted for our observations (compared to GVRD 9:00 am).

2. The concentration of MBAS in the effluent at the Lions Gate WWTP appears to follow a consistent diurnal cycle, with relatively low concentrations of 2-4 mg/L during the mid morning, increasing to 6 mg/L by mid afternoon, and increasing further to 8-12 mg/L by late evening or early morning. The observed diurnal cycle in MBAS concentration may be caused by use of clothes washers and dishwashers by residents during the evening.

3. The samples of primary effluent taken soon after midnight at the Lions Gate WWTP consistently contained MBAS concentrations in the range 8 mg/L to 10 mg/L, which is well in excess of the toxicity threshold of 2 mg/L to 2.5 mg/L MBAS tentatively identified by others.

4. Fish mortality in the treated samples from the batch tests was not directly related to MBAS concentration. In three of the six batch tests, samples that were found to be non-acutely toxic according to the bioassay had higher MBAS concentrations than samples that were acutely toxic within the same batch test. During our batch tests, the concentration of MBAS did not appear to be a reliable indicator of fish toxicity, nor did the greatest degree of MBAS removal appear to result in the greatest improvement in toxicity testing results.

5. The analysis for MBAS encompasses a large number of individual compounds, which may have varying degrees of toxicity, and which may be removed in different amounts by different treatment processes. This could account for the apparent inconsistencies in the MBAS results discussed above. Further study is needed to identify individual MBAS compounds, their toxicity, and the effectiveness of biological versus chemical treatment in reducing the most toxic compounds to non-toxic levels.

6. As far as MBAS removal is concerned, the most effective of the processes tested was 50% biological treatment (average 57%) removal, followed by CEP+25% biological treatment (44% removal). The least effective was CEP alone (26% removal). Removal of MBAS to maintain the concentration in the effluent consistently below the 2.5 mg/L toxicity threshold identified by others would probably require the implementation of full secondary treatment.

7. Chemically enhanced primary treatment (CEP) followed by 25% biological treatment was the most effective of all treatments tested in reducing the frequency of toxicity at Lions Gate (83% improvement compared to control). This was much better than the two next best toxicity reduction processes (50% biological and CEP alone) at 33% improvement.

8. Chemically enhanced primary treatment followed by 25% biological treatment was also the most effective process tested at Lions Gate (except for 100% biological treatment) in removal of TBOD and SBOD, and was near the top in removal of TSS and MBAS (50% biological treatment was the top process for TSS and MBAS removal).

9. For interim treatment at Lions Gate, CEP+25% biological treatment appears to be the most promising of the processes tested if toxicity reduction is the priority. If reduction of chemical parameters (TSS, BOD and MBAS) is the priority, 50% biological treatment appears to be slightly better than CEP+25% biological treatment.

6 RECOMMENDATIONS

The following recommendations are based on the findings of this study.

1. From the standpoint of reducing effluent toxicity, chemically enhanced primary treatment, 50% biological treatment, and CEP followed by 25% biological treatment should all be considered for interim upgrades at the Iona Island WWTP.

2. From the standpoint of reducing effluent toxicity, the focus for interim upgrades at the Lions Gate WWTP should be on chemically enhanced primary treatment followed by partial (e.g., 25%) biological treatment.

3. Additional study is needed to evaluate the toxicity of individual anionic surfactants contained in the influent to the Lions Gate WWTP, and the removal rates of those surfactants by chemical and biological processes (this is currently being undertaken by UBC Civil Engineering).

4. Further recommendations are included in Appendices 3 and 10 based on modeling and economic analysis.

7 REFERENCES

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APPENDIX A: IONA ISLAND WWTP BATCH TEST RESULTS

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #1 - Chemical Testing, Tues. Aug. 5/03 **(Note - for CEP polymer dose was 200 mg/L for Test #1 only)**

Test effluent taken at 9:00 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.1	7.1	22.0	22.0	35		65		49		14.5		2.0		N/A
CEP	6.9		22.0		40	0%	35	46%	25	49%	12.4	14%	1.4	30%	N/A
25% Bio	7.7	7.5	22.5		30	14%	49	25%	35	29%	13.8	5%	1.5	25%	674
50% Bio	7.7	7.5	22.5		27	23%	34	48%	21	57%	14.4	1%	1.1	45%	674
CEP+25% Bio	7.0		22.0		62	0%	32	51%	21	57%	14.8	0%	1.0	50%	706

Test #2 - Chemical Testing, Mon. Aug. 11/03

Test effluent taken at 8:15 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	6.8	6.8	22.5	22.5	26		69		55		12.5		2.7		N/A
CEP	6.3	6.8	22.5	22.7	9	65%	27	61%	25	55%	12.6	0%	2.6	4%	N/A
25% Bio	6.9	7.4	22.5	22.6	18	31%	47	32%	35	36%	13.3	0%	1.2	56%	554
50% Bio	6.9	7.4	22.5	22.6	11	58%	35	49%	23	58%	16.3	0%	1.5	44%	554
CEP+25% Bio	6.7	7.2	22.7	22.8	12	54%	21	70%	17	69%	13.9	0%	1.0	63%	517

Test #3 - Chemical Testing, Wed. Aug. 20/03

Test effluent taken at 10:05 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.2	7.2	22.1	22.1	41		117		78		18.4		1.4		N/A
CEP	6.7	7.0	22.3	22.4	18	56%	34	71%	33	58%	17.1	7%	0.9	36%	N/A
25% Bio	7.1	7.7	22.7	25.0	33	20%	60	49%	59	24%	13.2	28%	1.0	29%	619
50% Bio	7.1	7.7	22.7	25.0	27	34%	50	57%	24	69%	15.0	18%	0.6	57%	619
CEP+25% Bio	6.9	7.7	22.8	25.0	18	56%	24	79%	19	76%	18.3	1%	0.7	50%	396
100% Bio	7.1	7.7	22.7	25.0	7	83%	<10	91%	<10	87%	16.3	11%	0.2	86%	619

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #4 - Chemical Testing

Date Thursday August 28, 2003

Test effluent taken at 1:15 pm

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.1	7.1	22.7	22.7	47		93		61		25.3		0.9		N/A
CEP	6.8	6.7	22.8	22.9	22	53%	40	57%	36	41%	21.6	15%	2.0	0%	N/A
25% Bio	7.3	7.7	23.1	22.9	35	26%	72	23%	45	26%	25.0	1%	0.8	11%	500
50% Bio	7.3	7.7	23.1	22.9	24	49%	43	54%	27	56%	23.4	8%	0.7	22%	500
CEP+25% Bio	6.9	7.6	22.9	22.9	29	38%	36	61%	21	66%	25.3	0%	1.1	0%	328
100% Bio	7.3	7.7	23.1	22.9	7	85%	11	88%	<10	84%	21.8	14%	0.2	78%	500

Test #5 - Chemical Testing

Date Thursday, September 4, 2003

Test effluent taken at 1:00 pm

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.2	7.2	23.1	23.1	42		84		55		24.7		1.2		N/A
CEP	6.8	6.8	23.2	23.2	47	0%	46	45%	33	40%	20.7	16%	1.5	0%	N/A
25% Bio	7.6	23.4	7.8	23.0	36	14%	58	31%	33	40%	24.5	1%	1.2	0%	535
50% Bio	7.6	23.4	7.8	23.0	34	19%	38	55%	20	64%	23.9	3%	0.9	25%	535
CEP+25% Bio	7.2	23.3	7.7	23.3	43	0%	32	62%	20	64%	23.5	5%	1.1	8%	530
100% Bio	7.6	23.4	7.8	23.0											535

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #6 - Chemical Testing

Date Wednesday, September 10, 2003

Test effluent taken at 1:30 pm

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.13	7.13	21.8	21.8	49		53		33		14.2		0.8		N/A
CEP	6.75	6.66	21.8	21.9	24	51%	23	57%	13	61%	15.7	0%	1.4	0%	N/A
25% Bio	7.42	7.33	22.3	21.8	45	8%	42	21%	23	30%	16.3	0%	0.8	0%	573
50% Bio	7.40	7.30	22.3	21.8	35	29%	33	38%	13	61%	16.3	0%	0.7	13%	573
CEP+25% Bio	6.85	7.31	22.1	21.9	24	51%	15	72%	<10	70%	16.6	0%	1.3	0%	512
100% Bio	7.40	7.30	22.3	21.8	10	80%	<10	81%	<10	70%	17.5	0%	0.5	38%	573

Iona Overall Summary of Results

Treatment	Iona Percent Removal from Settled Sewage										Toxicity Removal
	TSS		TBOD		SBOD		NH3		MBAS		
	Avg	S Dev	Avg	S Dev	Avg	S Dev	Avg	S Dev	Avg	S Dev	
CEP	38%	30%	56%	10%	50%	9%	9%	7%	12%	17%	60%
25% Bio	19%	8%	30%	10%	31%	6%	6%	11%	20%	21%	20%
50% Bio	35%	15%	50%	7%	61%	5%	5%	7%	34%	17%	60%
CEP+25% Bio	33%	26%	66%	10%	67%	6%	1%	2%	29%	29%	60%
100% Bio	83%	3%	87%	5%	80%	9%	8%	7%	67%	26%	N/A

TSS TBOD SBOD MBAS TOXICITY

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #1 - Acute Toxicity Bioassay, 9:00 AM Tues. Aug. 5/03 **(Note - for CEP polymer dose was 200 mg/L for Test #1 only)**

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.1	7.5	18.4	12.3	1.6	6.0	0%	100%	Fail	< 1 hr
CEP	6.9	7.5	21.0	12.3	3.4	6.1	0%	100%	Fail	< 1 hr
25% Bio	7.1	7.9	18.4	6.8	3.6	9.6	100%	100%	Pass	Pass
50% Bio	7.1	7.9	18.4	6.8	3.4	9.4	100%	100%	Pass	Pass
CEP+25% Bio	7.1	7.7	18.4	9.3	3.8	9.2	100%	100%	Pass	Pass
Disinfected	7.1	8.1	18.4	4.5	4.2	7.0	0%	100%	Fail	< 1 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #2 - Acute Toxicity Bioassay, 8:15 AM Mon. Aug. 11/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.1	7.5	18.4	12.3	2.9	6.1	0%	100%	Fail	< 24 hr
CEP	6.9	7.6	21.0	10.8	6.2	8.8	100%	100%	Pass	Pass
25% Bio	7.2	7.7	16.9	9.3	3.2	7.8	0%	100%	Fail	< 24 hr
50% Bio	7.3	7.8	15.4	8.0	5.4	8.5	100%	100%	Pass	Pass
CEP+25% Bio	7.1	7.7	18.4	9.3	6.6	9.0	100%	100%	Pass	Pass
Disinfected	7.6	8.7	10.8	1.3	5.4	7.2	0%	100%	Fail	< 24 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #3 - Acute Toxicity Bioassay, 10:15 AM Wed. Aug. 20/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.2	7.6	16.9	10.8	1.8	6.4	0%	100%	Fail	< 1 hr
CEP	7.2	7.7	16.9	9.3	4.6	8.6	100%	100%	Pass	Pass
25% Bio	7.2	7.7	16.9	9.3	2.2	7.1	0%	100%	Fail	< 1 hr
50% Bio	7.2	7.9	16.9	6.8	4.1	8.2	10%	100%	Pass	Pass
CEP+25% Bio	7.2	7.7	16.9	9.3	5.6	9.0	100%	100%	Pass	Pass
Disinfected	7.2	7.8	16.9	8.0	3.8	7.0	0%	100%	Fail	< 2 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #4 - Acute Toxicity Bioassay

Date Thursday August 28, 2003

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	7.2	19.7	16.9	1.6	0.4	0%	100%	Fail	< 1 hr
CEP	6.7	7.5	23.0	12.3	4.4	8.7	70%	100%	Pass	Pass
25% Bio	7.1	7.4	18.4	13.9	2.6	6.0	0%	100%	Fail	< 1 hr
50% Bio	7.1	7.4	18.4	13.9	2.8	6.0	0%	100%	Fail	< 1 hr
CEP+25% Bio	6.9	7.4	21.0	13.9	6.4	7.4	20%	100%	Fail	< 24 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #5 - Acute Toxicity Bioassay

Date Thursday, September 4, 2003

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	N/A	19.7	N/A	1.6	N/A	0%	90%	Fail	< 1 hr
CEP	7.0	7.5	19.7	12.3	4.7	8.8	10%	100%	Fail	< 24 hr
25% Bio	7.2	7.4	16.9	13.9	2.0	7.5	10%	90%	Fail	< 24 hr
50% Bio	7.3	7.8	15.4	8.0	5.2	8.0	10%	100%	Fail	< 24 hr
CEP+25% Bio	7.0	7.5	19.7	12.3	5.4	8.8	40%	90%	Fail	< 24 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

APPENDIX 1: GVRD FACILITIES PLANNING: IONA ISLAND WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

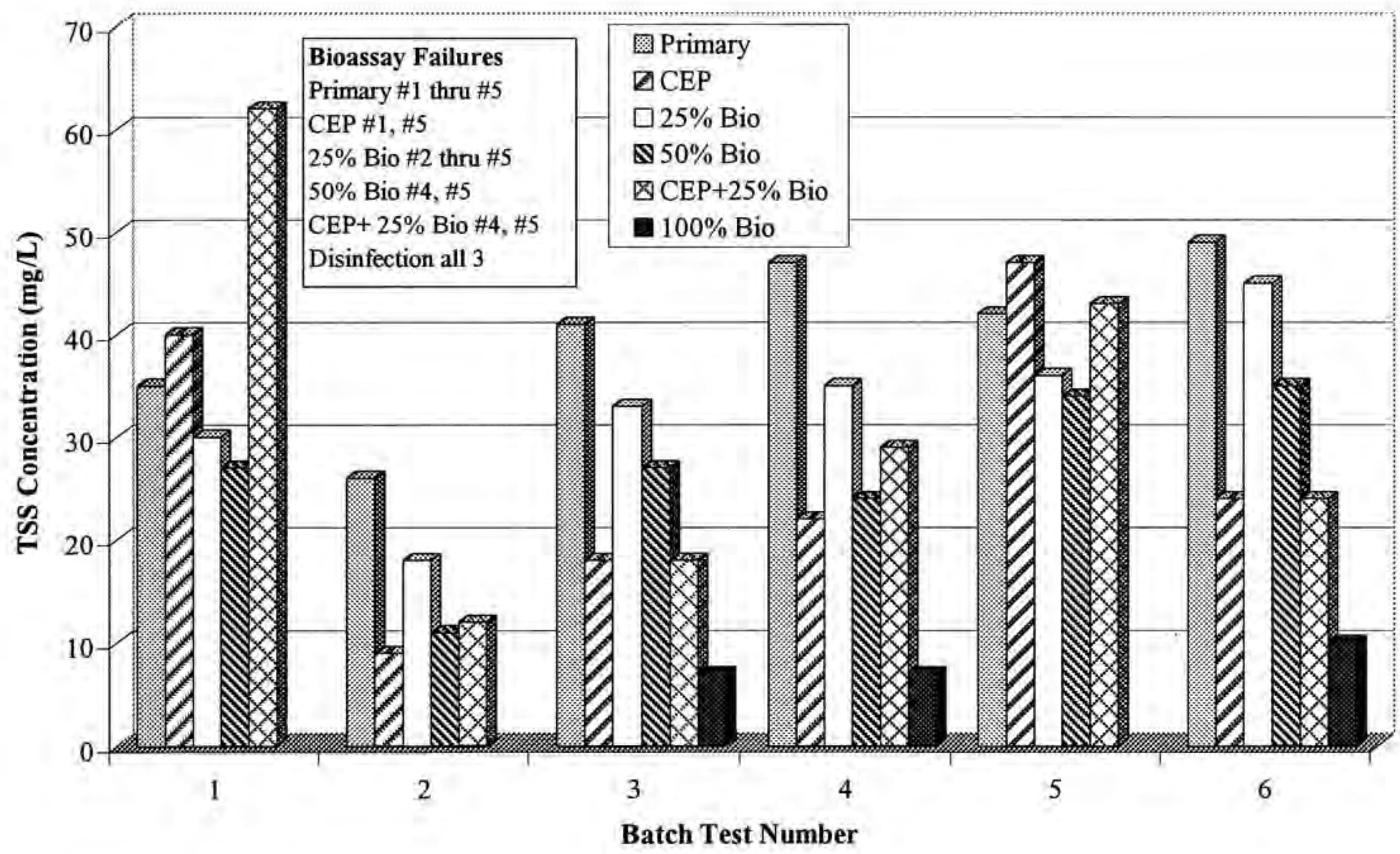
Test #6 - Acute Toxicity Bioassay

Date Wednesday, September 10, 2003

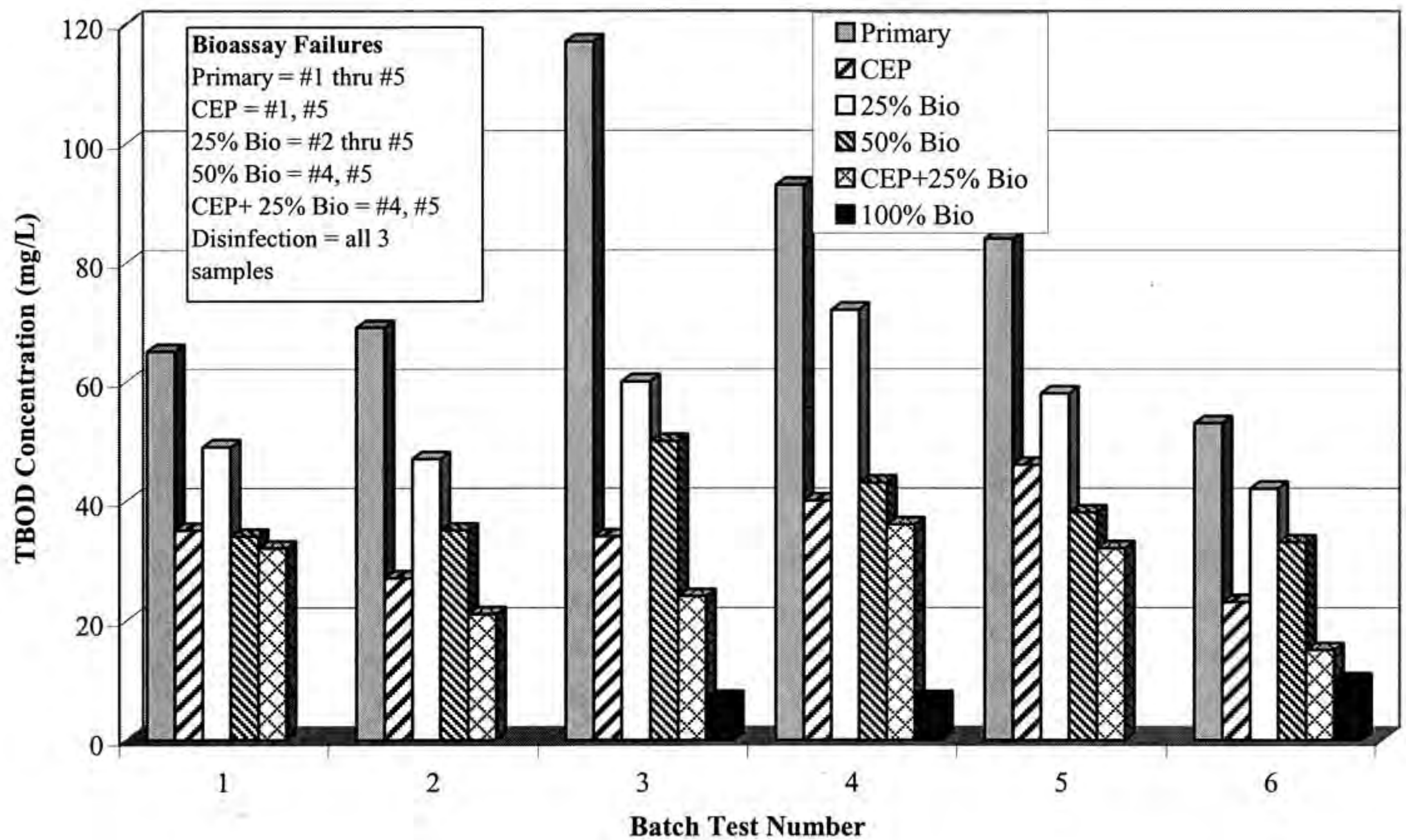
Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.1	7.3	18.4	15.4	4.8	7.4	80%	N/A	Pass	Pass
CEP	6.9	7.2	21.0	16.9	4.7	8.9	100%	N/A	Pass	Pass
25% Bio	7.1	7.2	18.4	16.9	3.5	6.6	80%	N/A	Pass	Pass
50% Bio	7.2	7.4	16.9	13.9	4.4	8.5	100%	N/A	Pass	Pass
CEP+25% Bio	6.9	7.2	21.0	16.9	5.7	8.0	100%	N/A	Pass	Pass

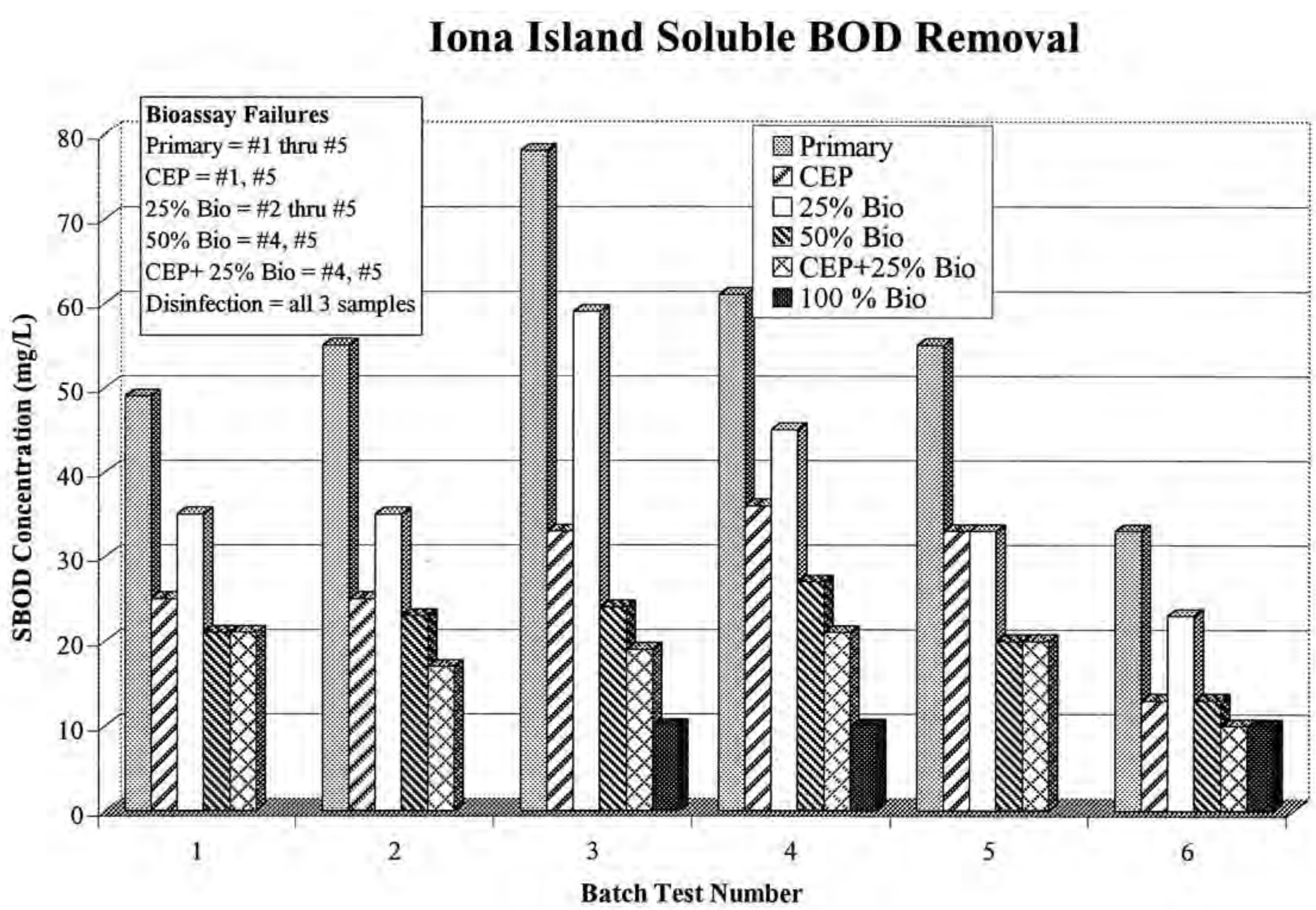
[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Iona Island TSS Removal

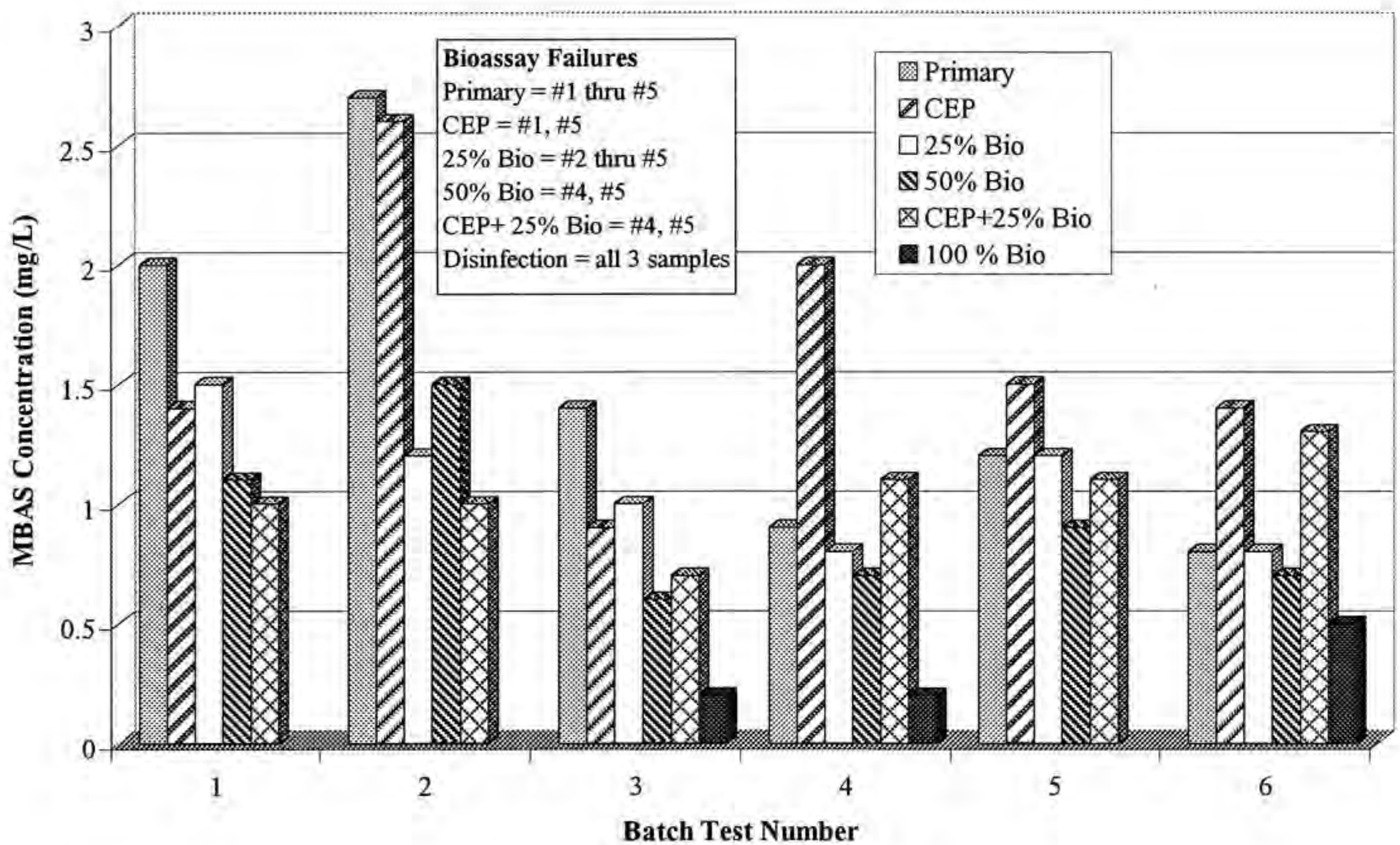


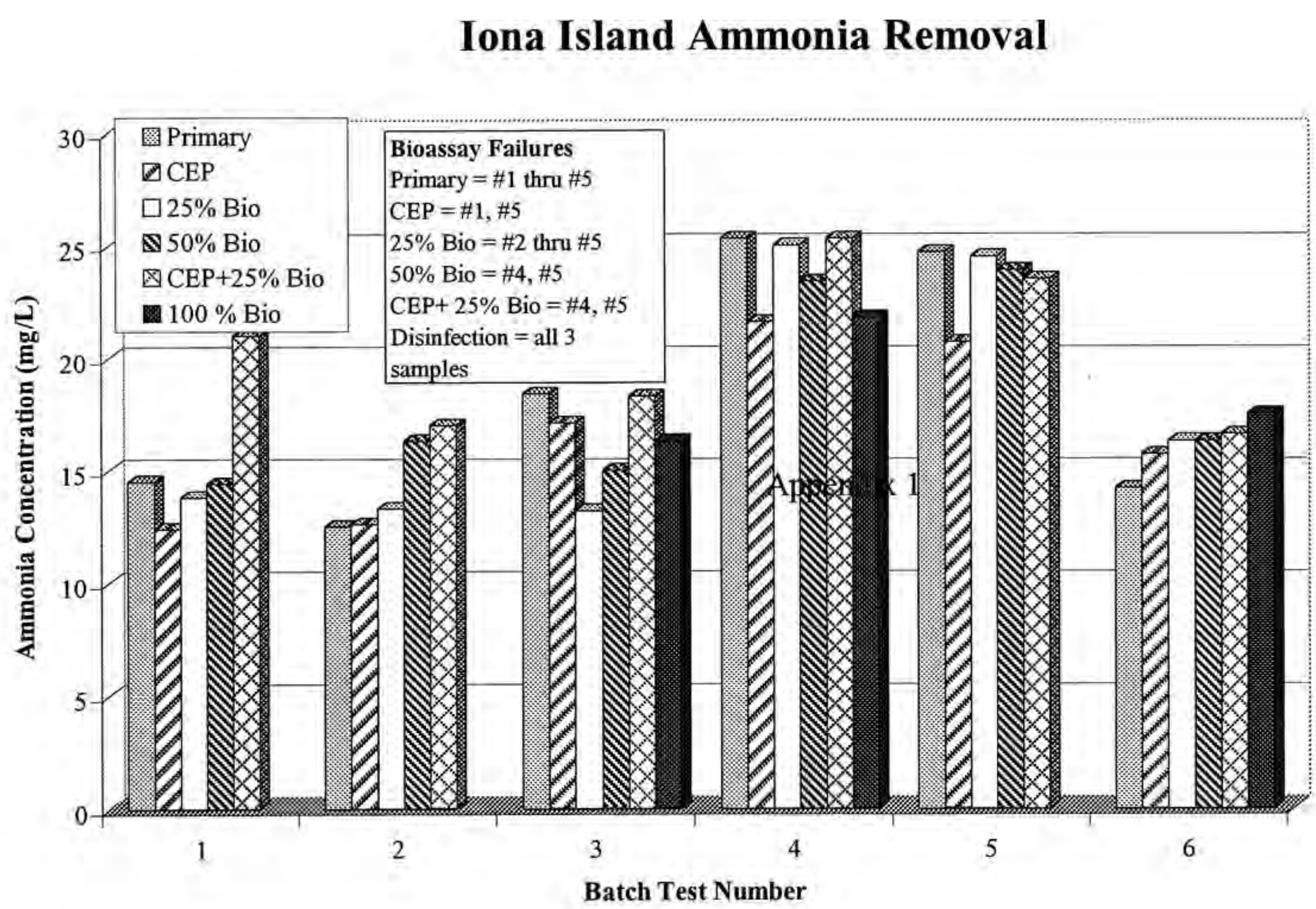
Iona Island TBOD Removal




Iona Island Soluble BOD Removal
Bioassay Failures
Primary = #1 thru #5
CEP = #1, #5
25% Bio = #2 thru #5
50% Bio = #4, #5
CEP+ 25% Bio = #4, #5
Disinfection = all 3 samples
Primary
CEP
25% Bio
50% Bio
CEP+25% Bio
100 % Bio
SBOD Concentration (mg/L)
0
10
20
30
40
50
60
70
80
1
2
3
4
5
6
Batch Test Number

Iona Island MBAS Removal




Iona Island Ammonia Removal
Primary
CEP
25% Bio
50% Bio
CEP+25% Bio
100 % Bio
Bioassay Failures
Primary = #1 thru #5
CEP = #1, #5
25% Bio = #2 thru #5
50% Bio = #4, #5
CEP+ 25% Bio = #4, #5
Disinfection = all 3
samples
Ammonia Concentration (mg/L)
0
5
10
15
20
25
30
1
2
3
4
5
6
Batch Test Number

APPENDIX B: LIONS GATE WWTP BATCH TEST RESULTS

APPENDIX 2: GVRD FACILITIES PLANNING: LIONS GATE WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #1 - Chemical Testing, Mon. Aug. 18/03

Test effluent taken at 12:05 am Test started at 8:00 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	6.8	6.8	20.7	20.7	56		117		87		20.7		8.3		NA
CEP	6.5	6.6	22.2	22.2	101	0%	95	19%	48	45%	17.0	18%	4.8	42%	NA
25% Bio	6.9	7.5	21.1	20.8	45	20%	90	23%	59	32%	19.6	5%	5.6	33%	630
50% Bio	6.9	7.5	21.1	20.8	32	43%	58	50%	34	61%	19.3	7%	3.5	58%	630
CEP+25% Bio	6.7	7.5	21.8	20.6	82	0%	69	41%	31	64%	19.9	4%	3.2	61%	600
100% Bio	6.9	7.5	21.1	20.8	6	89%	<10	91%	<10	89%	19.5	6%	1.2	86%	630

Test #2 - Chemical Testing, Thurs. Aug. 21/03

Test effluent taken at 12:05 am Test started at 8:25 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	6.8	6.8	25.0	25.0	60		142		103		15.9		12.1		NA
CEP	6.5	6.6	25.0	25.0	37	38%	70	51%	62	40%	15.8	1%	3.7	69%	NA
25% Bio	7.0	7.5	25.0	20.4	48	20%	86	39%	75	27%	16.1	0%	6.6	45%	683
50% Bio	7.0	7.5	25.0	25.0	38	37%	62	56%	46	55%	15.2	4%	4.3	64%	683
CEP+25% Bio	6.7	7.5	21.0	20.4	33	45%	51	64%	47	54%	19.4	0%	5.9	51%	716
100% Bio					5	92%	<10	93%	<10	90%	14.5	9%	0.4	97%	683

Test #3 - Chemical Testing, Wed. Aug. 27/03

Test effluent taken at 12:05 am Test started at 8:00 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	7.0	7.0	20.6	20.6	30		101		67		22.2		8.9		NA
CEP	6.6	6.6	20.6	21.3	22	27%	34	66%	25	63%	21.4	4%	6.2	30%	NA
25% Bio	7.0	7.5	20.2	20.0	37	0%	58	43%	46	31%	22.0	1%	5.3	40%	672
50% Bio	7.0	7.5	20.2	20.0	11	63%	46	54%	30	55%	22.6	0%	3.9	56%	672
CEP+25% Bio	6.9	7.6	21.3	20.2	3	90%	29	71%	17	75%	23.4	0%	3.8	57%	724
100% Bio	7.0	7.5	20.2	20.0	5	83%	12	88%	<10	85%	20.5	8%	0.2	98%	672

APPENDIX 2: GVRD FACILITIES PLANNING: LIONS GATE WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #4 - Chemical Testing, Tues. Sept. 2/03

Test effluent taken at 12:05 am   Test started at 8:30 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	6.9	6.9	19.8	19.8	54		131		86		22.5		9.3		NA
CEP	6.6	6.6	20.2	20.9	47	13%	79	40%	56	35%	21.3	5%	7.8	16%	NA
25% Bio	7.1	7.6	20.4	19.8	40	26%	93	29%	59	31%	24.9	0%	6.4	31%	531
50% Bio	7.1	7.6	20.4	19.8	32	41%	54	59%	37	57%	23.9	0%	2.9	69%	531
CEP+25% Bio	6.9	7.3	21.1	25.0	38	30%	57	56%	34	60%	24.1	0%	6.5	30%	544

Test #5 - Chemical Testing, Tues. Sept 9/03

Test effluent taken at 12:30 am   Test started at 1:00 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	6.95	6.95	21.4	21.4	50		118		76		24.6		5.8		NA
CEP	6.72	6.72	21.2	20.6	35	30%	50	58%	35	54%	22.6	8%	6.7	0%	NA
25% Bio	7.16	7.60	21.4	19.0	43	14%	88	25%	51	33%	23.3	5%	4.3	26%	462
50% Bio	7.16	7.60	21.4	19.0	39	22%	55	53%	28	63%	23.6	4%	3.3	43%	462
CEP+25% Bio	6.87	7.57	20.4	18.1	34	32%	42	64%	22	71%	23.7	4%	3.9	33%	543

Test #6 - Chemical Testing, Tues. Sept. 23/03

Test effluent taken at 12:05 am   Test started at 8:00 am

Reactor	pH		Temperature		TSS		TBOD		SBOD		Ammonia		MBAS		MLSS
	Initial	Final	Initial	Final	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)	% rem.	(mg/L)
Untreated	N/A	N/A	19.4	19.4	47		99		77		25.6		8.0		NA
CEP	N/A	N/A	19.0	19.2	22	53%	45	55%	40	48%	23.6	8%	8.1	0%	NA
25% Bio	N/A	N/A	19.2	19.2	38	19%	86	13%	57	26%	25.8	0%	6.9	14%	764
50% Bio	N/A	N/A	19.2	19.2	43	9%	52	47%	34	56%	24.6	4%	3.8	53%	764
CEP+25% Bio	N/A	N/A	19.4	19.0	27	43%	33	67%	25	68%	26.9	0%	5.6	30%	730

APPENDIX 2: GVRD FACILITIES PLANNING: LIONS GATE WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Lions Gate Overall Summary of Results

Treatment	Lions Gate Percent Removal from Settled Sewage										Toxicity Removal
	TSS		TBOD		SBOD		NH3		MBAS		
	Avg	S Dev	Avg	S Dev	Avg	S Dev	Avg	S Dev	Avg	S Dev	
CEP	27%	19%	48%	17%	47%	10%	7%	6%	26%	27%	33%
25% Bio	16%	9%	29%	11%	30%	3%	2%	3%	32%	11%	1%
50% Bio	36%	19%	53%	4%	58%	3%	3%	3%	57%	9%	33%
CEP+25% Bio	40%	29%	61%	11%	65%	7%	1%	2%	44%	14%	83%
100% Bio	88%	4%	91%	2%	88%	3%	7%	2%	93%	7%	N/A

TSS TBOD SBOD MBAS TOXICITY

APPENDIX 2: GVRD FACILITIES PLANNING: LIONS GATE WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #1 - Acute Toxicity Bioassay, 12:05 AM Mon. Aug. 18/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.1	7.3	18.4	15.4	3.0	2.0	0%	100%	Fail	< 2 hr
CEP	6.9	7.4	21.0	13.9	3.4	8.1	0%	100%	Fail	< 24 hr
25% Bio	7.2	7.4	16.9	13.9	2.4	3.4	0%	100%	Fail	< 2 hr
50% Bio	7.4	7.7	13.9	9.3	2.6	6.8	40%	100%	Fail	< 48 hr
CEP+25% Bio	7.1	7.6	18.4	10.8	3.6	7.4	70%	100%	Pass	Pass

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #2 - Acute Toxicity Bioassay, 12:05 AM Thurs. Aug. 21/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	7.7	19.7	9.3	5.0	7.1	0%	100%	Fail	< 24 hr
CEP	7.5	7.8	12.3	8.0	5.2	8.6	90%	100%	Pass	Pass
25% Bio	7.2	NA	16.9	N/A	3.6	NA	0%	100%	Fail	< 2 hr
50% Bio	7.6	7.8	10.8	8.0	6.2	7.6	100%	100%	Pass	Pass
CEP+25% Bio	7.0	8.0	19.7	5.7	4.2	9.6	100%	100%	Pass	Pass

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #3 - Acute Toxicity Bioassay, 12:05 AM Wed. Aug. 27/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.1	7.1	18.4	18.4	2.0	3.6	0%	100%	Fail	< 1 hr
CEP	7.2	7.6	16.9	10.8	6.3	8.9	10%	100%	Fail	< 24 hr
25% Bio	7.3	7.4	15.4	13.9	3.5	6.4	0%	100%	Fail	< 48 hr
50% Bio	7.4	7.5	13.9	12.3	3.8	7.4	90%	100%	Pass	Pass
CEP+25% Bio	7.4	7.5	13.9	12.3	6.8	9.0	100%	100%	Pass	Pass

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

APPENDIX 2: GVRD FACILITIES PLANNING: LIONS GATE WWTP SMALL-SCALE TESTING

CEP - Chemically-Enhanced Primary Treatment, 70 mg/L alum + 0.5 mg/L anionic polymer (NALCO 1C34)

Bio - Biological Treatment using waste biological sludge from Annacis WWTP

Test #4 - Acute Toxicity Bioassay, 12:05 AM Tues. Sept. 2/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	N/A	19.7	N/A	3.0	N/A	0%	40%	Fail	< 1 hr
CEP	6.9	7.2	21.0	16.9	5.8	6.8	90%	100%	Pass	Pass
25% Bio	7.1	N/A	18.4	N/A	3.6	N/A	0%	100%	Fail	< 2 hr
50% Bio	7.2	7.5	16.9	12.3	3.0	6.6	30%	100%	Fail	< 24 hr
CEP+25% Bio	7.0	7.3	19.7	15.4	3.4	6.2	40%	100%	Fail	< 72 hr

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #5 - Acute Toxicity Bioassay, 12:30 AM Tues. Sept. 9/03

Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	7.5	19.7	12.3	4.4	7.0	0%	30%	Fail	< 4 hr
CEP	6.9	7.4	21.0	13.9	6.4	7.8	20%	60%	Fail	< 48 hr
25% Bio	6.8	7.4	22.0	13.9	3.5	8.7	0%	90%	Fail	< 24 hr
50% Bio	7.2	7.6	16.9	10.8	4.4	7.6	20%	100%	Fail	< 24 hr
CEP+25% Bio	7.1	7.6	18.4	10.8	4.4	9.0	50%	90%	Pass	Pass

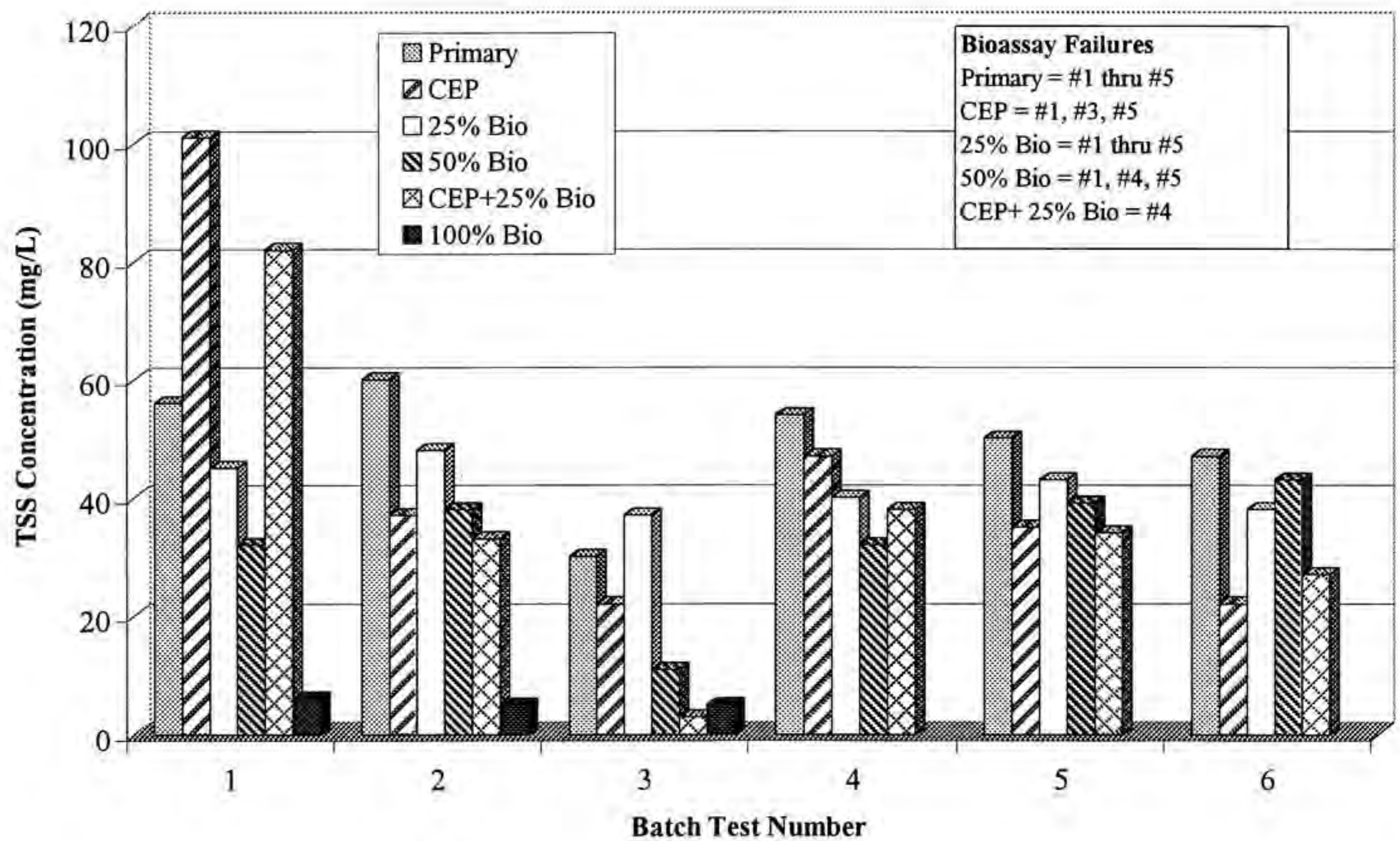
[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

Test #6 - Acute Toxicity Bioassay, 12:05 AM Tues. Sept. 23/03

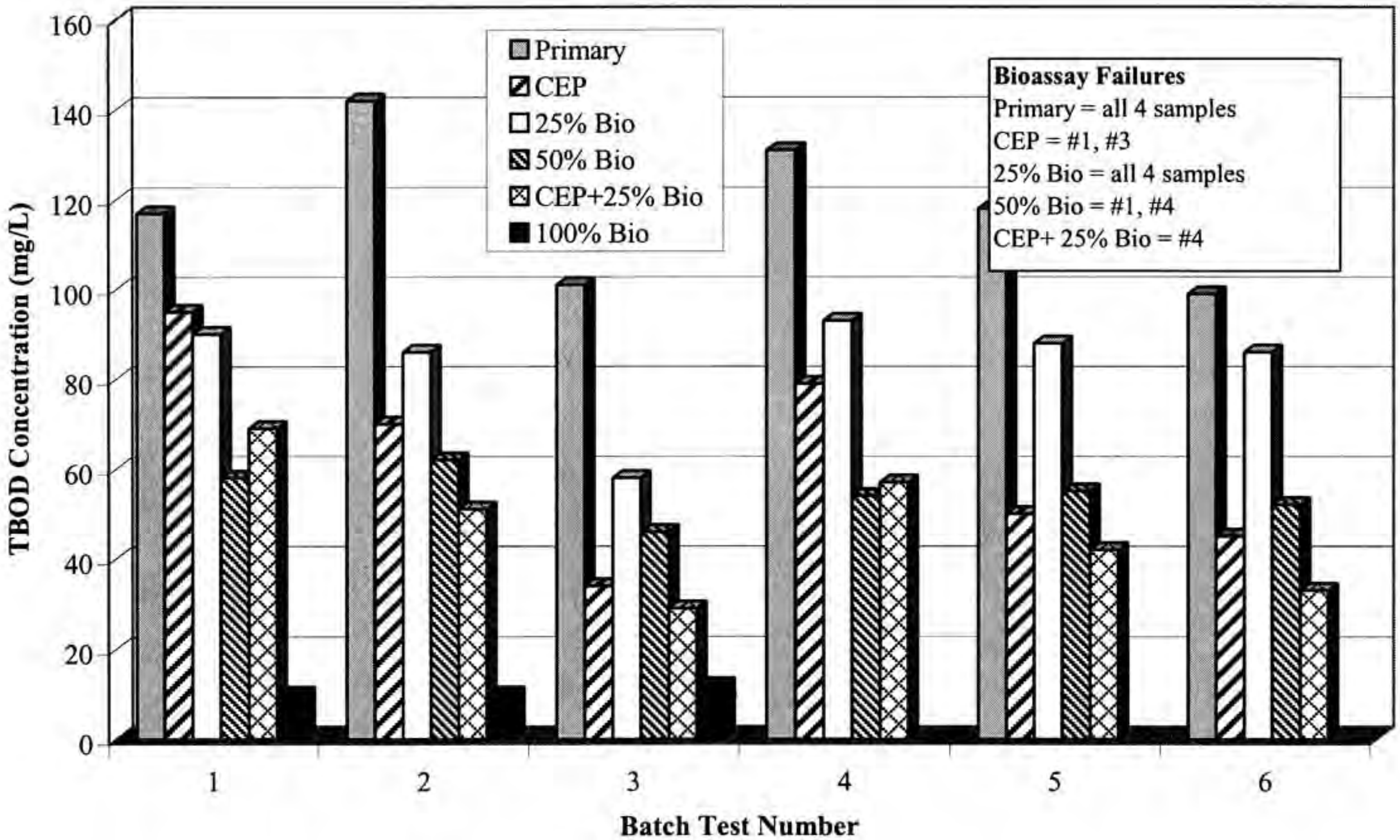
Reactor	pH @100% Concentration		NH_3 Tox. Threshold (mg/L) @100% Conc.[1]		Dissolved Oxygen Conc. (mg/L) @100% Conc.		% Fish Survival @ 96 hours		Pass/Fail	Time to Fail
	t=0 hr	t=24 hr	t=0 hr	t=24 hr	t=0 hr	t=24 hr	100% Conc.	50% Conc.		
Untreated	7.0	N/A	19.7	N/A	2.1	N/A	0%	N/A	Fail	< 1 hr
CEP	6.8	7.0	22.0	19.7	3.2	4.6	0%	N/A	Fail	< 24 hr
25% Bio	7.1	N/A	18.4	N/A	1.0	N/A	0%	N/A	Fail	< 1 hr
50% Bio	7.3	N/A	15.4	N/A	1.8	N/A	0%	N/A	Fail	< 1 hr
CEP+25% Bio	7.1	7.3	18.4	15.4	4.0	6.6	80%	N/A	Pass	Pass

[1] from BC Water Quality Criteria tables for acute ammonia toxicity at 15 degrees C, based on ammonia conc. @ t=0 (ammonia conc. @ t=24 unknown)

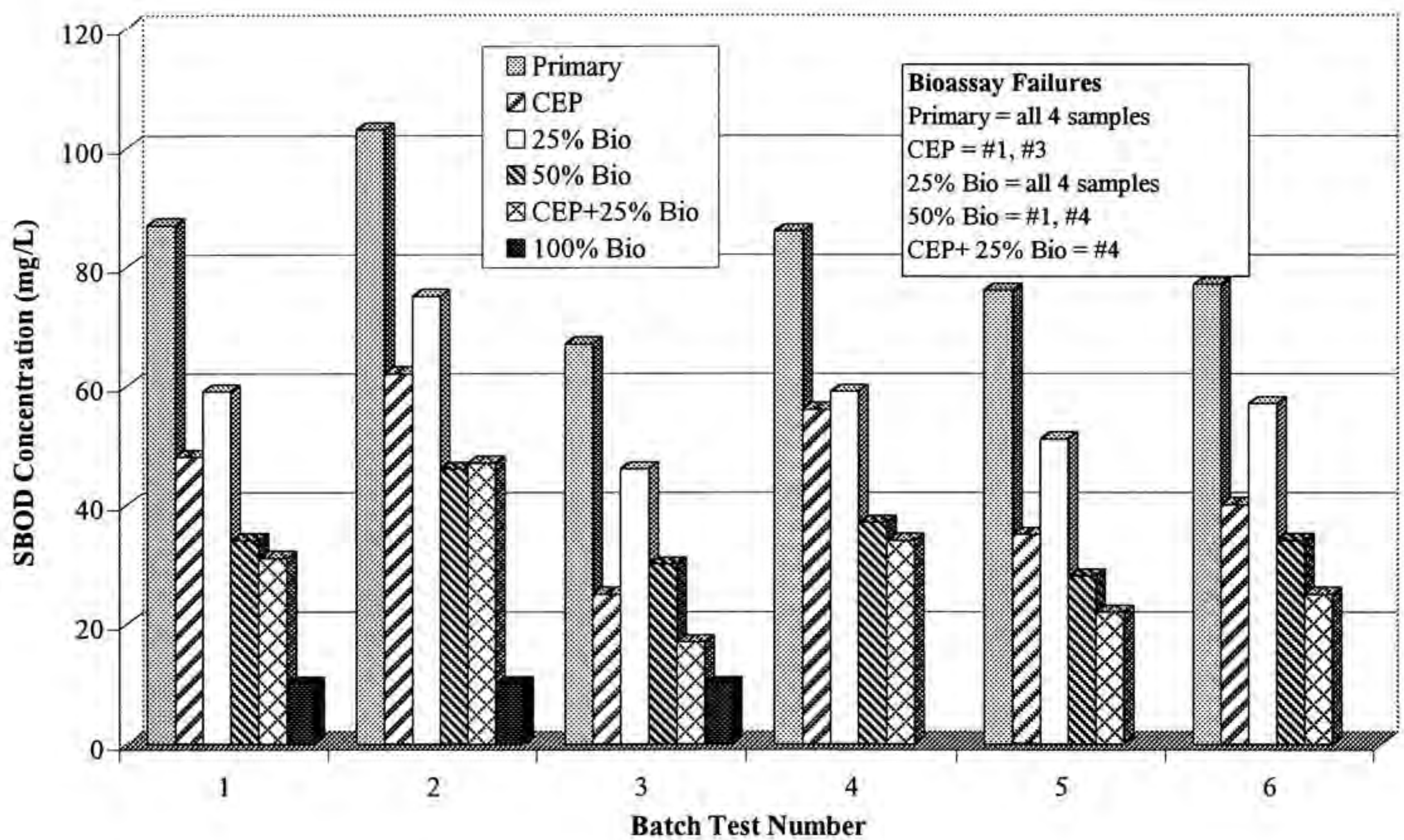
Lions Gate TSS Removal



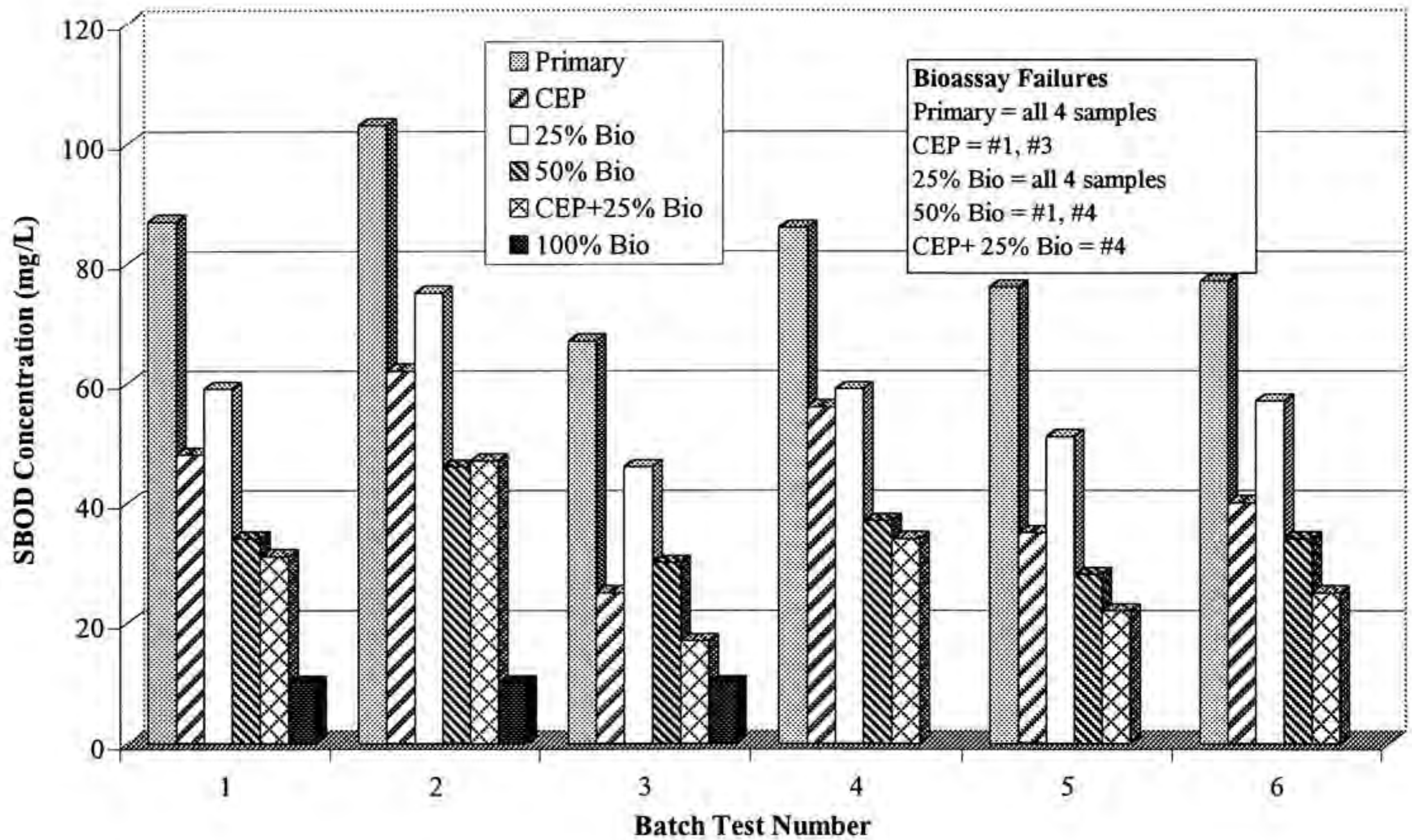
Lions Gate TBOD Removal



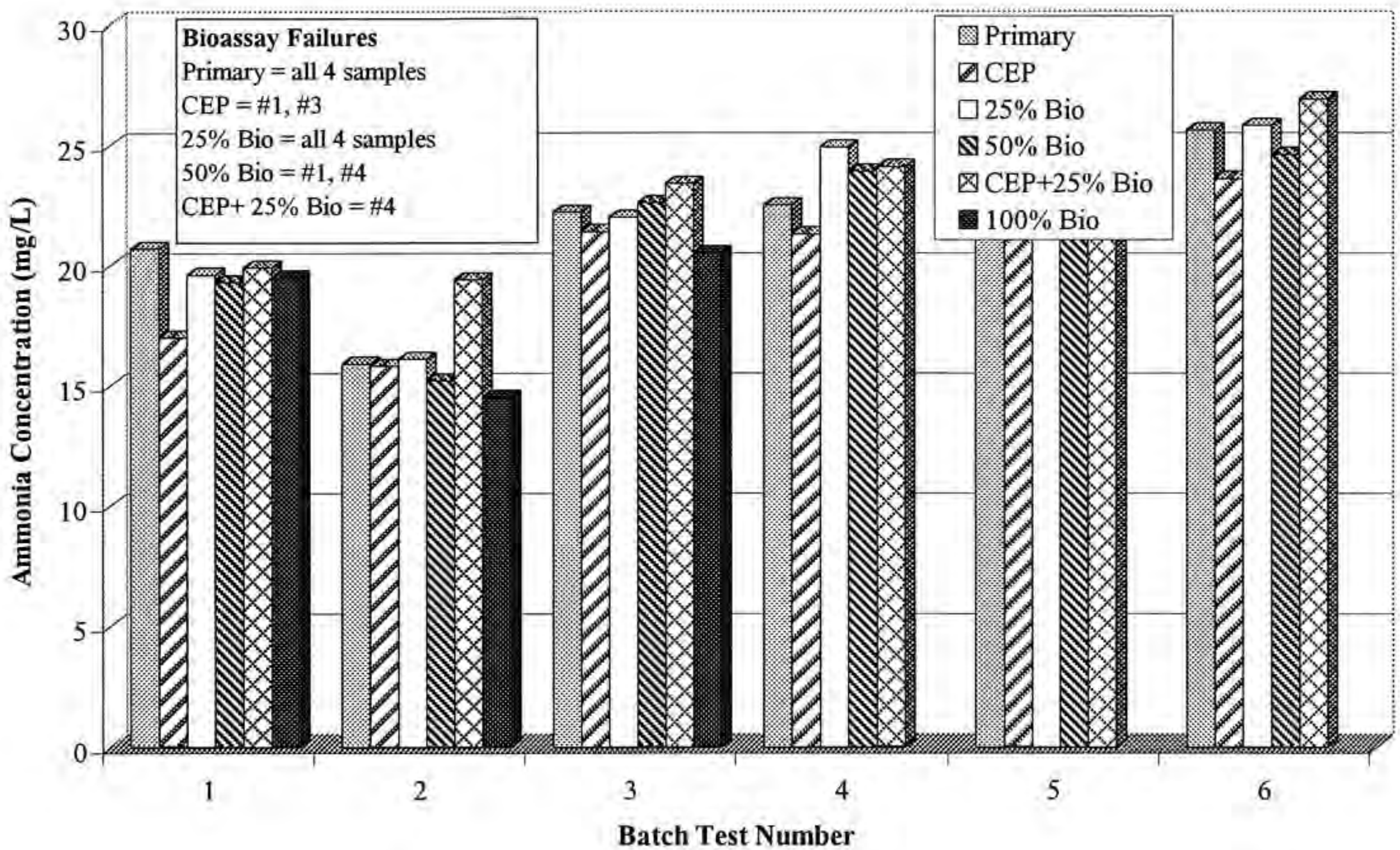
Lions Gate Soluble BOD Removal



Lions Gate Soluble BOD Removal



Lions Gate Ammonia Removal



APPENDIX C: SUMMARY COMPARISON OF BATCH TEST RESULTS FOR IONA ISLAND AND LIONS GATE WWTP'S

APPENDIX 3: GVRD FACILITIES PLANNING: IONA ISLAND AND LIONS GATE WWTP SMALL-SCALE TESTING SUMMARY OF RESULTS

Summary of Acute Toxicity Bioassays

Treatment	Iona (96 hr LC50)		Lions Gate (96 hr LC50)	
	Pass	Fail	Pass	Fail
Primary (Control)	1	5	0	6
CEP	4	2	2	4
25% Bio	2	4	0	6
50% Bio	4	2	2	4
CEP+25% Bio	4	2	5	1
Disinfection	0	3	N/A	N/A

Summary of Removal of Chemical Parameters

Treatment	Average Percent Removal				
	TSS	TBOD	SBOD	NH3	MBAS
CEP Iona	38%	56%	50%	9%	12%
CEP LG	27%	48%	47%	7%	26%
25% Bio Iona	19%	30%	31%	6%	20%
25% Bio LG	16%	29%	30%	2%	32%
50% Bio Iona	35%	50%	61%	5%	34%
50% Bio LG	36%	53%	58%	3%	57%
CEP+25% Bio Iona	33%	66%	67%	1%	29%
CEP+25% Bio LG	40%	61%	65%	1%	44%
100% Bio Iona	83%	87%	80%	8%	67%
100% Bio LG	88%	91%	88%	7%	93%

Iona TSS Results

Treatment	TSS Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	35	26	41	47	42	49
CEP	40	9	18	22	47	24
25% Bio	30	18	33	35	36	45
50% Bio	27	11	27	24	34	35
CEP+25% Bio	62	12	18	29	43	24
100% Bio			7	7		10

Lions Gate TSS Results

Treatment	TSS Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	56	60	30	54	50	47
CEP	101	37	22	47	35	22
25% Bio	45	48	37	40	43	38
50% Bio	32	38	11	32	39	43
CEP+25% Bio	82	33	3	38	34	27
100 % Bio	6	5	5			

Iona TBOD Results

Treatment	TBOD Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	65	69	117	93	84	53
CEP	35	27	34	40	46	23
25% Bio	49	47	60	72	58	42
50% Bio	34	35	50	43	38	33
CEP+25% Bio	32	21	24	36	32	15
100% Bio			10	11		10

Lions Gate TBOD Results

Treatment	TBOD Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	117	142	101	131	118	99
CEP	95	70	34	79	50	45
25% Bio	90	86	58	93	88	86
50% Bio	58	62	46	54	55	52
CEP+25% Bio	69	51	29	57	42	33
100 % Bio	10	10	12			

APPENDIX 3: GVRD FACILITIES PLANNING: IONA ISLAND AND LIONS GATE WWTP SMALL-SCALE TESTING SUMMARY OF RESULTS

Iona SBOD Results

Treatment	SBOD Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	49	55	78	61	55	33
CEP	25	25	33	36	33	13
25% Bio	35	35	59	45	33	23
50% Bio	21	23	24	27	20	13
CEP+25% Bio	21	17	19	21	20	10
100 % Bio			10	10		10

Lions Gate SBOD Results

Treatment	SBOD Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	87	103	67	86	76	77
CEP	48	62	25	56	35	40
25% Bio	59	75	46	59	51	57
50% Bio	34	46	30	37	28	34
CEP+25% Bio	31	47	17	34	22	25
100 % Bio	10	10	10			

Iona MBAS Results

Treatment	MBAS Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	2	2.7	1.4	0.9	1.2	0.8
CEP	1.4	2.6	0.9	2.0	1.5	1.4
25% Bio	1.5	1.2	1.0	0.8	1.2	0.8
50% Bio	1.1	1.5	0.6	0.7	0.9	0.7
CEP+25% Bio	1	1	0.7	1.1	1.1	1.3
100 % Bio			0.2	0.2		0.5

Lions Gate MBAS Results

Treatment	MBAS Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	8.3	12.1	8.9	9.3	5.8	8.0
CEP	4.8	3.7	6.2	7.8	6.7	8.1
25% Bio	5.6	6.6	5.3	6.4	4.3	6.9
50% Bio	3.5	4.3	3.9	2.9	3.3	3.8
CEP+25% Bio	3.2	5.9	3.8	6.5	3.9	5.6
100 % Bio	1.2	0.4	0.2			

Iona Ammonia Results

Treatment	Ammonia Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	14.5	12.5	18.4	25.3	24.7	14.2
CEP	12.4	12.6	17.1	21.6	20.7	15.7
25% Bio	13.8	13.3	13.2	25.0	24.5	16.3
50% Bio	14.4	16.3	15.0	23.4	23.9	16.3
CEP+25% Bio	21	17	18.3	25.3	23.5	16.6
100 % Bio			16.3	21.8		17.5

Lions Gate Ammonia Results

Treatment	Ammonia Concentration (mg/L)					
	Test #1	Test #2	Test #3	Test #4	Test #5	Test #6
Primary	20.7	15.9	22.2	22.5	24.6	25.6
CEP	17	15.8	21.4	21.3	22.6	23.6
25% Bio	19.6	16.1	22.0	24.9	23.3	25.8
50% Bio	19.3	15.2	22.6	23.9	23.6	24.6
CEP+25% Bio	19.9	19.4	23.4	24.1	23.7	26.9
100 % Bio	19.5	14.5	20.5			

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 6
Diversion of North Shore
to Iona Island

FINAL REPORT

Prepared for

Greater Vancouver Regional District

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August 2005

117-00018

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1 INTRODUCTION

1.1 OVERVIEW

This phase of the Facility Plan involves developing conceptual plans based on the possible diversion of flows to the Iona Island Wastewater Treatment Plant (IIWWTP) from the North Shore. The feasibility of a marine pipeline crossing with a pumping facility located at the existing Lions Gate Wastewater Treatment Plant (LGWWTP) or alternate North Shore location is assessed. An examination of a range of flow diversion scenarios and North Shore wet weather options are reviewed, from full diversion to diversion of only dry weather flow.

1.2 SCOPE OF WORK

The Scope of work is summarized from the RFP as follows:

- Review existing relevant reports.
- Examine a range of flow and loading projections from only dry weather flow to complete diversion of wet weather flow.
- Identify potential sites for the required pumping facilities, including relocation of the works.
- Identify and evaluate at least the following marine pipeline crossings routes:

  a) Across Burrard Inlet west around Point Grey, tying in to the headworks of the IIWWTP.

  b) Across Burrard Inlet/English Bay tying into the Highbury Interceptor at 1st Avenue.

  c) Similar to a) except parallel this route with a new tunnel or pump station forcemain combination to the IIWWTP. Consider O&M benefits.
- Consider the alternative of maintaining a modified LGWWTP as a wet weather plant or siting a new wet weather facility on the North Shore.
- Assess impact on IIWWTP from the additional flow from LGWWTP.
- Marine crossing options are to consider:

  a) research in other jurisdictions regarding design and O&M requirements.

  b) research of previous Burrard Inlet crossings.

  c) geotechnical requirements.

  d) pipe size, material and corrosion resistance.

  e) seismic requirements.

f) use of a two pipe system.

g) pipeline cleaning requirements.

h) chemical injection requirements.

- Approval requirements.
- Identification of short and long term effects of flow diversion in CSO's and SSO's.
- Develop a short list of options and provide conceptual design drawings and descriptions for review.

2 OBJECTIVE

The objective of this part of the study is to determine if it is feasible to consider changing the LGWWP into a pumping station site. Under this arrangement, flows of up to 2 x ADWF would be pumped to the IIWWTP and wet weather flows above 2 x ADWF would be treated at the LGWWTP.

3 DESIGN CRITERIA

3.1 FLOW AND LOAD

The Design Flow and Load Criteria are as summarized near the end of this memorandum in Table 1.

For the LGWWTP, upgrading to full secondary treatment is required by the year 2030. Since most treatment facilities are designed for a 50-year life projection, the year 2081 was selected as the appropriate design year for the LGWWTP. Using flow and load projections detailed elsewhere in this study, the 2081, 2 x ADWF is assessed at 280 ML/d. The Annual Average Flow (AAF) value is 1.2 x the ADWF.

TABLE 1
DESIGN FLOW AND LOAD CRITERIA

			IONA ISLAND WWTP			LIONS GATE WWTP			
			Existing (2002)	Year 2021	Year 2036	Existing (2002)	Year 2031	Year 2046	Year 2081
	Population	[3]U	-	751,193	773,583	-	242,841	283,610	341,733
		[3]B	621,793	720,522	755,000	173,750	215,849	248,002	321,169
		[3]L	-	700,596	710,114	-	215,578	241,146	289,861
FACTORS	AAF / ADWF		1.34			1.2			
	MDF / ADWF		3.1			2.4			
	PWWF / ADWF	U	-	3.70	3.69	-	3.26	3.21	3.15
		B	3.75	3.71	3.70	3.38	3.30	3.25	3.17
		L	-	3.72	3.71	-	3.30	3.26	3.20
	MML / AAL		1.31 (BOD) 1.38 (TSS)			1.34 (BOD) 1.43 (TSS)			
FLOW	ADWF (ML/d)	U	-	502	516	-	116	131	150
		B	436	483	500	91 (Permit Value: 102 ML/d)	105	117	140
		L	-	414	383	-	90	91	91
	2 x ADWF (ML/d)	U	-	1004	1032	-	232	262	300
		B	872	966	1000	180	210	234	280
		L	-	828	766	-	180	182	182
	PWWF ([2]PHF) (ML/d)	U	-	1856	1904	-	378	420	472
		B	1634	1791	1848	307	346	380	443
		L	-	1538	1422	-	297	297	291
	Flow Ceiling (ML/d)		1530 (Permit ***& Capacity of Exist Primary)***			300 (Influent Pump Capacity)			
LOAD	[4]AA BOD (t/d)	U	-	95	98	-	19	22	26
		B	78	90	95	13	16	19	25
		L	-	81	81	-	16	17	19
	[1]MM BOD (t/d)	U	-	124	128	-	26	30	35
		B	102	118	124	18	22	26	33
		L	-	106	106	-	21	23	26
	[4]AA TSS (t/d)	U	-	81	85	-	22	25	29
		B	67	77	82	15	19	22	28
		L	-	72	72	-	17	19	22
	[1]MM TSS (t/d)	U	-	112	117	-	31	36	42
		B	93	107	113	22	27	31	40
		L	-	100	99	-	25	27	31

Note: [1]BOD & TSS loading from Iona Island WWTP exclude Trucked Liquid Waste.
[2]Peak Hour Flow (PHF) = AAF * [1+ (14/(4+(Population in 1000s)$^{0.5}$)] + (MDF – AAF)
[3]U: Upper Envelope; B: Base Case; L: Lower Envelope
[4]AA is back calculated from MM / (Peaking Factor of MML / AAL). Values not derived in spreadsheets.

3.2 NUMBER OF FORCEMAINS

A twin pipeline system needs to be considered. Advantages of a twin pipeline system are:

- Provides redundancy in the event of failure of one of the pipes.
- Allows maintenance on one pipeline while the other one is in service.
- Additional cost is relatively small.
- During the earlier design years, it will be easier to minimize grit deposition in the forcemain by only using one of the pipelines.

A twin-pipe system is preferred by GVRD's Operations staff.

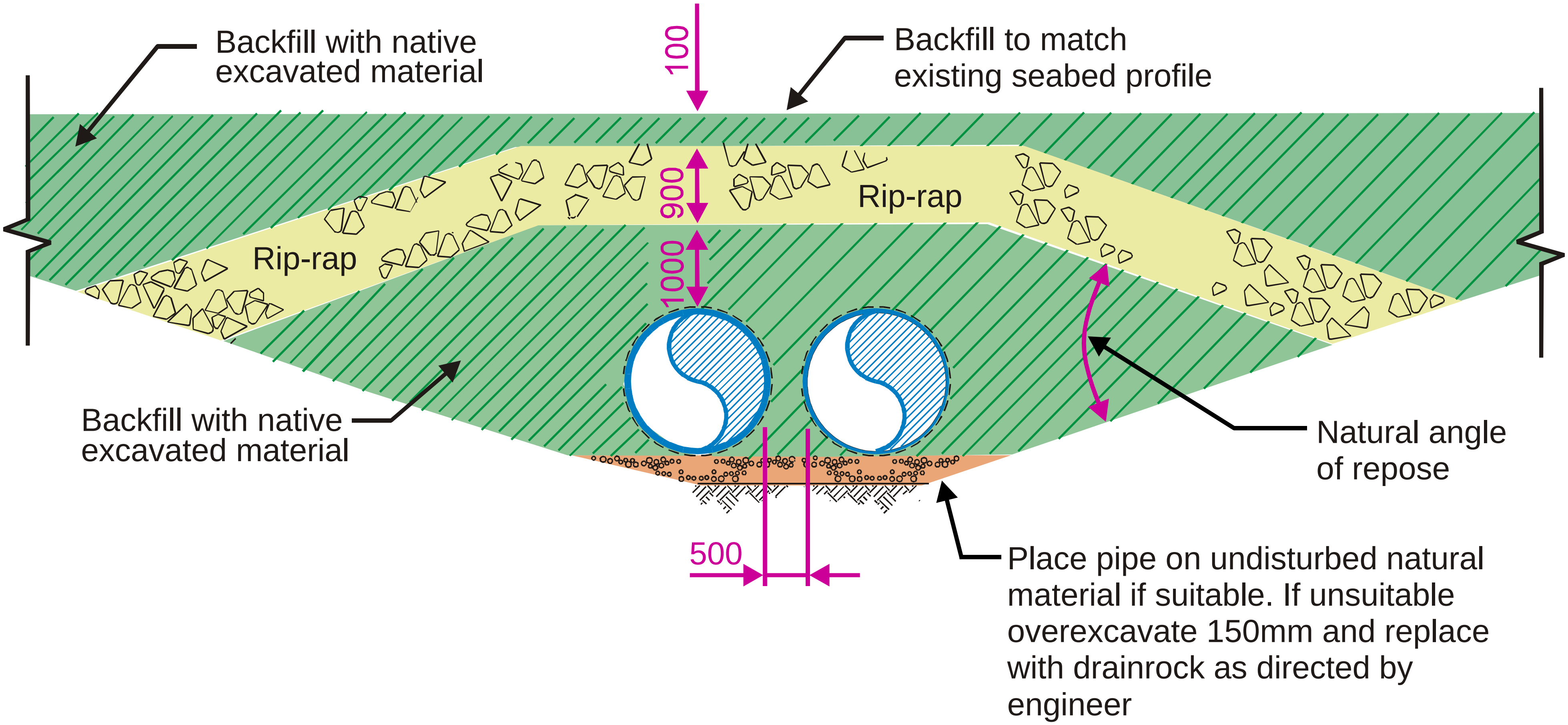
3.3 DIAMETER SELECTION

When assessing a pumped system, it is essential that a proper balance occur between the pump horsepower and the friction head in the forcemain. Typically design criteria used by the GVRD and most consultants utilize an energy gradient between 0.004 to 0.006. We have selected 0.005 for this project. Forcemain velocities typically range from 2 m/sec up to a maximum of 3 m/sec.

Friction losses increase exponentially with velocity so that higher velocities significantly increase energy and O&M costs as well as increase the costs for pump selection and pump control. For a discharge of 280 ML/d and a velocity of 2.5 m/sec, a 1,270 mm (50 inch) pipeline is required for a hydraulic slope of 0.005. For a twin pipe system, two 965 mm (38 inch) pipelines would be necessary with a velocity of 2.2 m (sec for a hydraulic slope of 0.005).

3.4 BURIAL DEPTH

Where the pipelines would cross under surface waters, Trow Associates Inc. advises that the pipeline would require burial in a trench with a 2 metre minimum cover depth. A rip-rap blanket would also be required over the top of each pipeline (except in the North Arm of the Fraser where dredging occurs and rip rap cover is not allowed). A typical conceptual cross section detail is illustrated on Figure 1.



Concept Typical Section Through Forcemains

4 ALIGNMENT OPTIONS

The RFP requires that at least the following be considered.

Route 1) Across Burrard Inlet west around Point Grey, tying in to the headworks of the IIWWTP.

Route 2a) Across Burrard Inlet/English Bay tying into the Highbury Interceptor at 1st Avenue.

Route 2b) Similar to a) except parallel this route with a new tunnel or pump station forcemain combination to the IIWWTP.

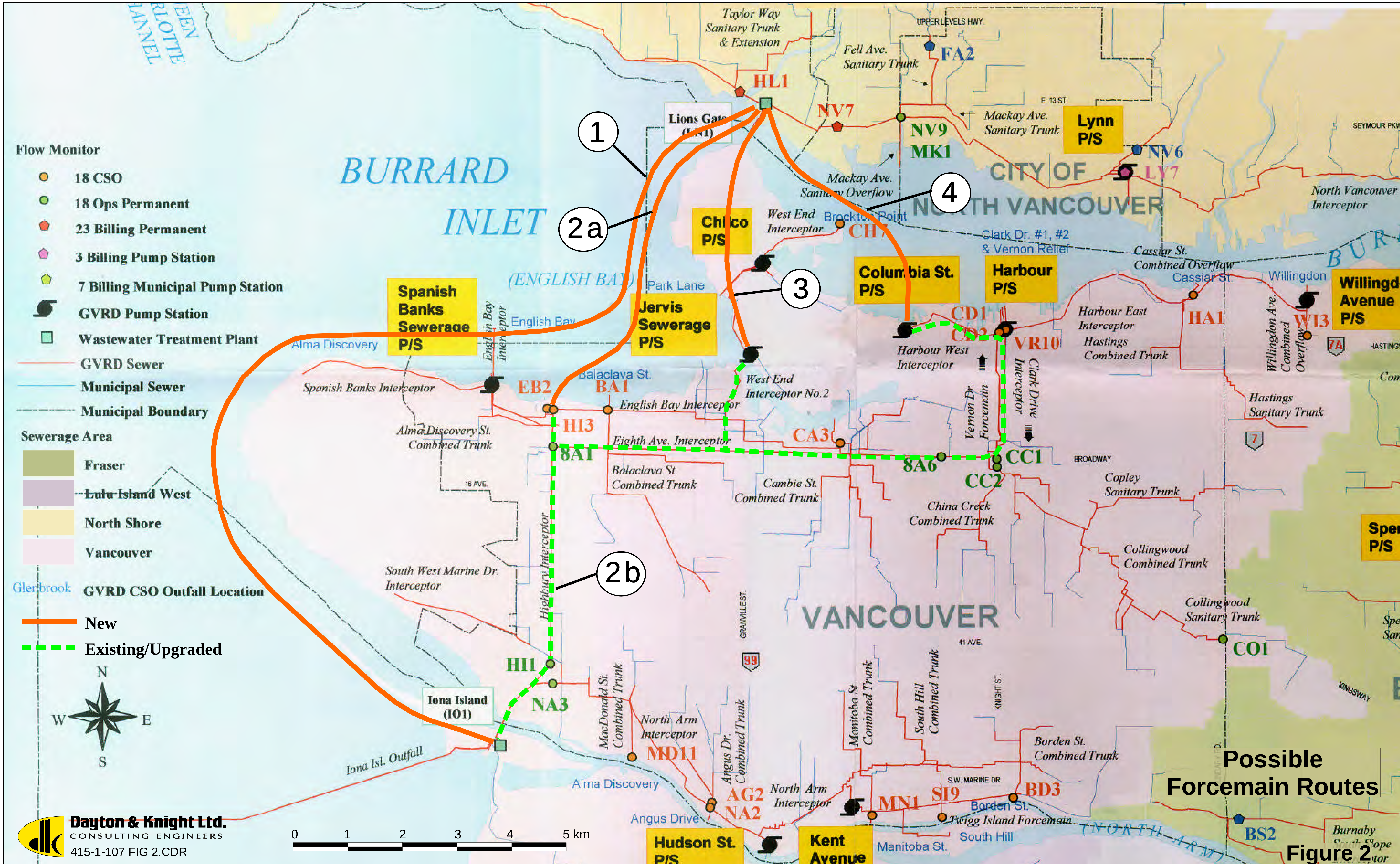
In addition, two other alignments had been considered by others:

Route 3) Under Stanley Park connecting to the Jervis Sewage Pumping Station and from there through an upgraded pumping system to the 8th Avenue Interceptor.

Route 4) Across the waters of Vancouver Harbour to the Columbia Street Pumping Station and from there through upgraded pumps to the 8th Avenue Interceptor.

A concept layout of these alternative routes is illustrated on Figure 2.

Following discussions at GVRD workshops, some further pipeline options were identified. These are discussed later in Section 7.



Possible Forcemain Routes

Figure 2

5 APPROVAL REQUIREMENTS

For any of the flow routes considered various approvals would be required. These approval agencies are summarized as follows:

- Squamish First Nation
- Burrard Environmental Review Committee (BERC)
    - Vancouver Port Authority (VPA)
    - Canadian Coast Guard (CCG)
    - Department of Fisheries and Oceans (DFO)
    - Environment Canada (EC)
    - Ministry of Water, Land and Air Protection (MWLAP)
- Possible Also
    - B.C. Environmental Assessment Act (Provincial)
    - Canadian Environmental Assessment Agency (CEAA)

In addition to the Approval requirements, there are various general obstacles that would require to be addressed, such as:

- Historic wrecks southwest of the midpoint of the narrows (Routes 1, 2a and 2b).
- Contaminated soils in First Narrows (mainly Routes 3 and 4).
- Requirements of Pacific Pilotage regarding anchorage (Routes 1, 2a and 2b).
- At least one fibre optic cable from the UBC area west to Vancouver Island (Route 1).
- Limitations on dredging and deposition in the North Arm (Route 1).
- Limitations on closure time for deep sea vessels (all Routes).
- Vancouver Aquarium intake pipes (Route 4).

Designated anchorage locations are illustrated on Figure 3.

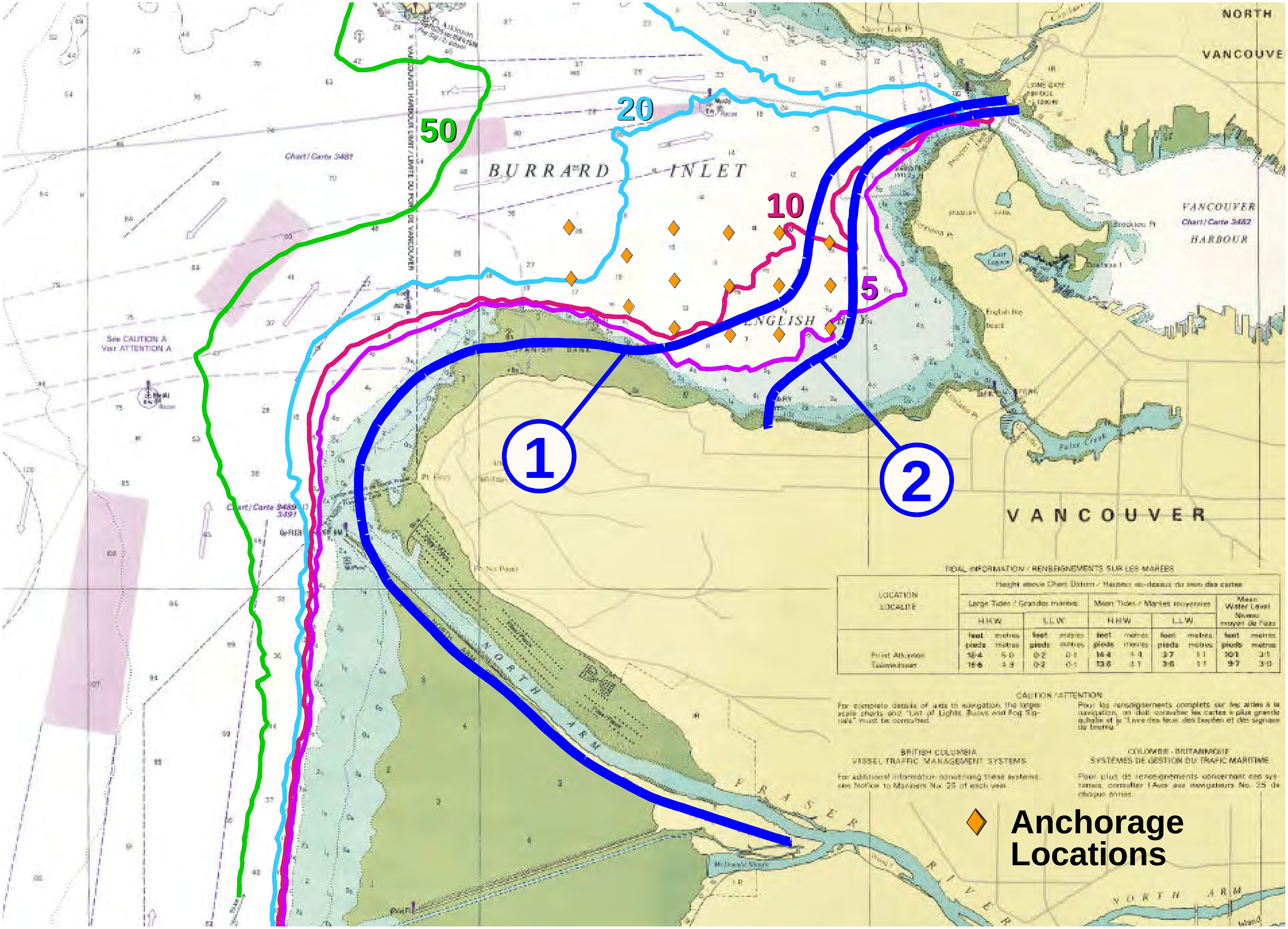

50
20
10
5
1
2
BURRARD INLET
ENGLISH BAY
NORTH VANCOUVER
VANCOUVER HARBOUR
VANCOUVER
NORTH ARM
FRASER RIVER
Anchorage Locations

Figure 3

6 FORCEMAIN MATERIALS

Various materials could be considered for a North Shore diversion forcemain. A general overview of the different materials follows:

6.1 CONCRETE

Advantages are its weight as no extra ballast would be required. A major disadvantage is the relatively short length of pipe sections making them relatively costly to install. Because of the relatively long length of forcemain, odour generation would be an issue and coating of the inside of the pipeline would be required.

6.2 FIBREGLASS

Advantages are high strength and excellent corrosion resistance. Distance between joints are greater than concrete but still a significant number of joints would exist. Additional ballasting would be necessary.

6.3 HIGH DENSITY POLYETHYLENE

Its' major advantages are no possible leaking joints would exist after fusion of the joints together with excellent corrosion resistance. A major disadvantage is its relatively low pressure capability for conveying pumped flows and the need for significant amounts of additional ballast (polyethylene is more buoyant than water).

6.4 STEEL

Its advantage is strength, no open joints after field welding has occurred and typically no need for additional buoyancy. Steel also has the best resistance to damage from anchors of the alternatives considered here. Disadvantages are the need for coating and possible deterioration from stray electrical currents.

Coal tar enamel for the coating and lining is usually the material of choice. The coating is usually protected by using lagging. Possible stray electrical currents can be addressed by installing cathodic protection systems with sacrificial anodes.

For this application we consider coated and lined steel pipe would offer the best selection for the forcemain material.

7 PRELIMINARY COST ESTIMATES

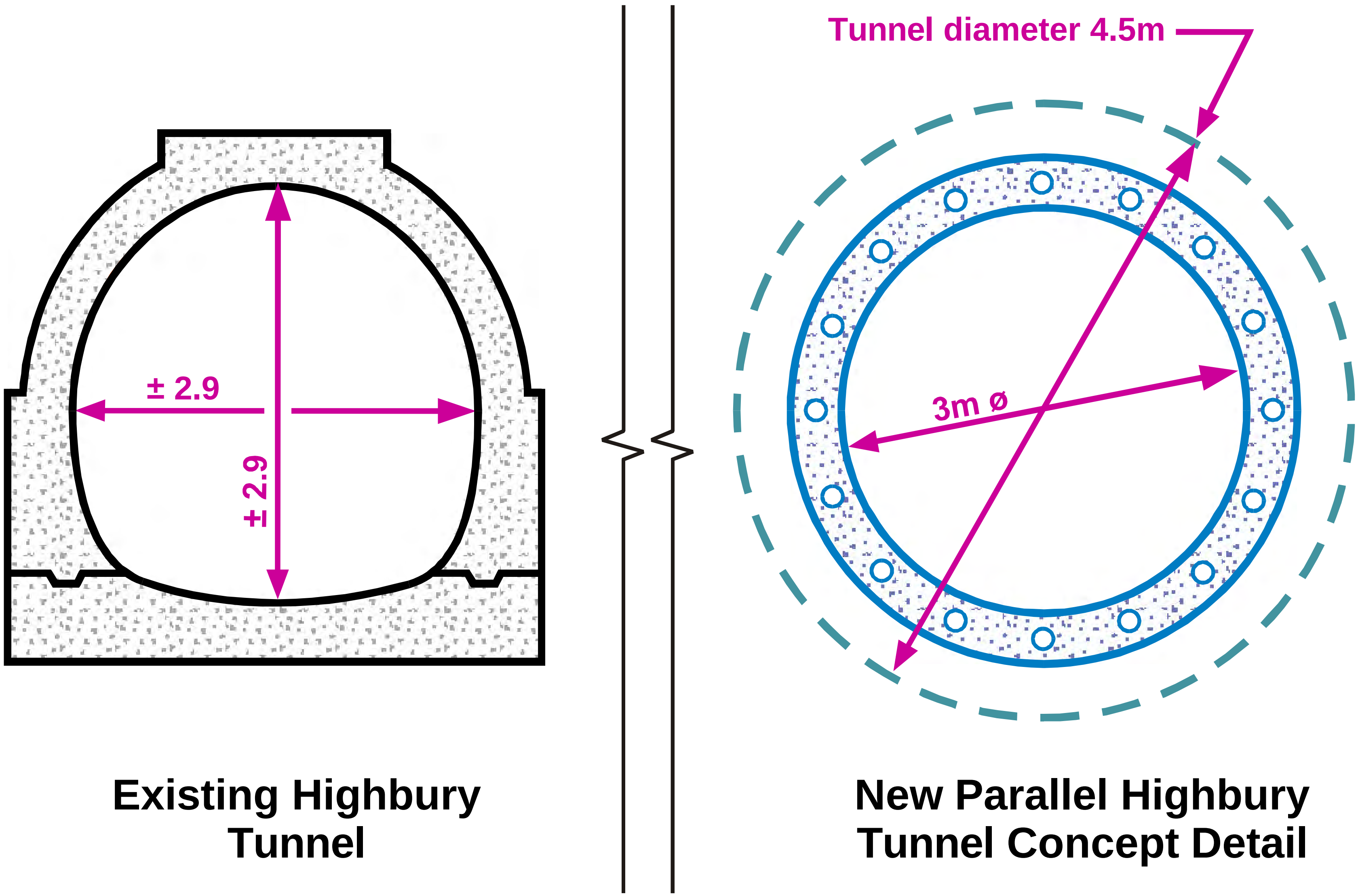
To assess whether or not Routes 3 and 4 were worth further consideration it was agreed to carry out a preliminary cost estimate of the four routes. In addition, the RFP required that Route 2 involve two sub-options as follows:

a) Pumping only to the start of the Highbury Interceptor.

b) Pumping to the Highbury Interceptor and from there through either the existing Interceptor or a new similar parallel one.

Figure 4 illustrates the existing Highbury Interceptor and a concept detail for the parallel Interceptor. From discussions with tunnel boring contractors, a 4.5 m tunnel would be required for a 3 m inside diameter parallel pipeline system such as is shown on Figure 4.

Table 2 summarizes the costs of Routes 1, 2a, 2b, 3 and 4 shown on Figure 2.



Existing Highbury Tunnel

New Parallel Highbury Tunnel Concept Detail

Figure 4

TABLE 2
NORTH SHORE FLOW DIVERSION TO IONA ISLAND WWTP CONCEPT COST ESTIMATES OF OPTIONS

Option	Marine Length (km)	Description	Concept Cost Estimates in Millions of Dollars					Total in Millions of Dollars	
			Lions Gate Pumping Station	Single Marine Pipeline	Twin Marine Pipeline	Single Land Pipeline	Twin Land Pipeline	Single Marine Pipeline	Twin Marine Pipeline
1	22.0	Marine pipeline from Lions Gate WWTP to Iona WWTP	33	48	60	--	--	**81**	**93**
2a	7.0	Marine Pipeline from Lions Gate WWTP to Highbury Interceptor	22	16.5	20.5	--	--	**38.5**	**42.5**
2b	13.5	Marine Pipeline from Lions Gate WWTP to Highbury Interceptor including twinning of interceptor	22	16.5	20.5	75	75	**113.5**	**117.5**
3	5.0	Tunnel below Burrard Inlet and Stanley Park to West End Interceptor No. 2	16.5	27.5	34	92.5*	93*	**136.5**	**143.5**
4	5.2	Marine pipeline from Lions Gate WWTP to Columbia Pump Station	16.5	15	19	113*	114.5*	**144.5**	**150**

* Costs include additional pumping station in Burrard Inlet, Forcemain upgrading to lift flow to 8th Avenue Interceptor, and Paralleling of both 8th Avenue and Highbury Interceptors.

Following a workshop meeting with GVRD the following was agreed:

- Routes 3 and 4 are to be abandoned from further consideration because of their relatively high cost.
- Option 2c, a 6 metre tunnel, is to be considered parallel to the Highbury Interceptor to provide additional benefits for reducing sewer overflows.
- As an alternative to paralleling the Highbury Interceptor, a shallow bury forcemain (single) is to be considered to convey:
  - Option 5a) 2 x ADWF from only the North Shore.
  - Option 5b) 2 x ADWF from the North Shore plus the ADWF from the 8^{th} Avenue Interceptor.

Present worth costs, including O&M consideration, and using a 6% discount factor were agreed as to be used. These costs are discussed further in Section 8.

In summary, five alignment options were identified to be studied, and refined concept estimates were obtained for these options.

All options commence their discharge from the Lions Gate Wastewater Treatment Plant.

The options presented provide alternate schemes, some of which would convey greater flows than others. In selecting a preferred option, consideration needs to be given to the magnitude of the cost of each option as part of the triple bottom line assessment.

These alignments are as follows:

7.1 OPTION 1

This option (see Figure 5) is identical to that of Route 2a. It consists of pumping the flow through two 965 mm diameter pipelines across English Bay to the northern end of the existing Highbury Interceptor.

7.2 OPTION 2

This option (see Figure 5) is identical to that of Route 2b. It consists of pumping the flow through two 965 mm diameter pipelines across English Bay to the northern end of the existing Highbury Interceptor. Flow would then be discharged into a new 3 metre diameter gravity tunnel parallel to the existing Highbury Interceptor.

Two geotechnical reports were reviewed entitled “Geology of Highbury Tunnel” prepared by Victor Dolmageia 1958 and 1959 before construction of the existing Highbury Interceptor. The reports note that from 22^{nd} Avenue south, large flows of artesian water were encountered in the drill holes in the sand, gravel, silt, clay and till. Some supplementary drilling reportedly proved that the artesian water was coming from a large sand aquifer 20 to 100 feet above the Highbury Tunnel elevation and separated from it by thick layers of impermeable till, clay and silt/clay strata. It was concluded by Dolmageia that since most of the drill samples recovered were at most damp, no extreme difficulties would likely be met driving the tunnel. It is our understanding that this was not the case and that dewatering requirements were almost overwhelming. For a tunnel parallel to the existing, the need for substantial dewatering is included in the cost estimates.

7.3 OPTION 3

This option (see Figure 5) is similar to that of Option 2 except that the new Highbury gravity tunnel would be 6 metres in diameter rather than 3 metres as in Option 2. The purpose of the larger tunnel would be to provide additional benefits for receiving combined sewer overflows from the 8^{th} Avenue Interceptor system.

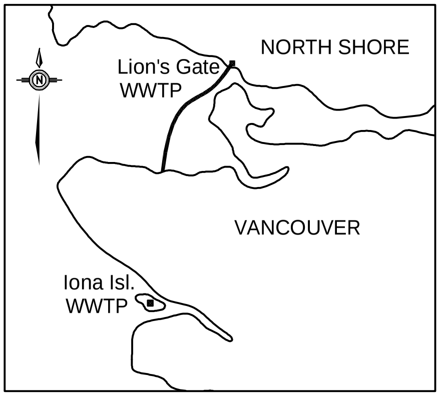
Data provided by GVRD on flows in the 8^{th} Avenue Interceptor are summarized in Appendix A. From that data:

- Daily maximum flow = 3.88 m^3/s
- Minimum flow = 2.08 m^3/s
- From hydrograph, ADWF = 3 to 3.53 m^3/s

Selecting 3.53 m^3/s = 305 ML/d

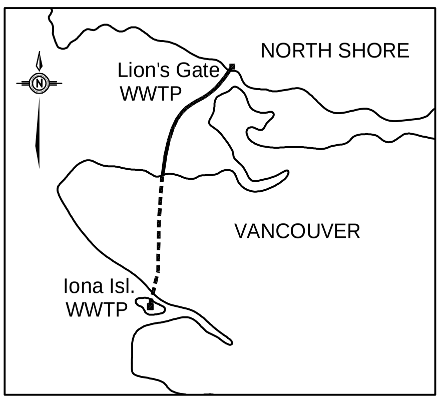
From ADWF data developed for Iona the ratio between ADWF for the study period 1999 to 2081 is 601/428 = 1.404.

The factored 8^{th} Avenue ADWF flow for 2081 then becomes 305 x 1.404 = 428 ML/d.



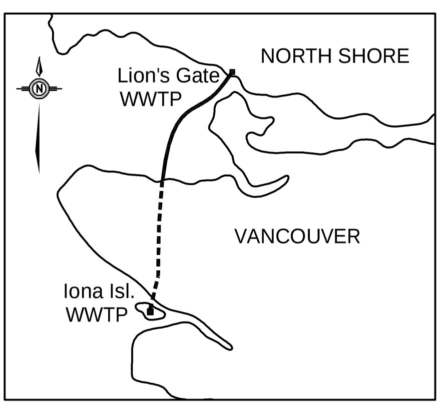
OPTION 1

- ENGLISH BAY ALIGNMENT OF TWO 965mm Ø PIPELINES TO THE START OF HIGHBURY INTERCEPTOR



OPTION 2

- ENGLISH BAY ALIGNMENT OF TWO 965mm Ø PIPELINES TO THE START OF HIGHBURY INTERCEPTOR
- PLUS 3m Ø TUNNEL PARALLEL TO EXISTING HIGHBURY INTERCEPTOR



OPTION 3

- ENGLISH BAY ALIGNMENT OF TWO 965mm Ø PIPELINES TO THE START OF HIGHBURY INTERCEPTOR
- PLUS 6m Ø TUNNEL PARALLEL TO EXISTING HIGHBURY INTERCEPTOR

POSSIBLE FORCEMAIN ROUTES
REFINED OPTIONS 1, 2, AND 3

ACAD DWG. FIGURE 5_6 1:1 04-01-13

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FIGURE 5

7.4 OPTION 4

Flows for this option (see Figure 6) are similar to those for Option 2b. Rather than paralleling the existing Highbury Interceptor with a new gravity tunnel, the North Shore Diversion flow would be pumped over the Point Grey Peninsula with a lift of about 91.5 metres (300 feet) through a single forcemain (1270 mm required). The costs of either a single pump station versus one low level station and one intermediate station were reviewed. At this level of study, the overall costs (capital plus O&M) were assessed to be very similar. We have therefore adopted considering only one station at this stage.

7.5 OPTION 5

Flows for this option (see Figure 6) are similar to those for Option 3. Option 5 considers the flows from the North Shore combined with those from 8th Avenue into a single forcemain (1800 mm diameter) and a single pumping station over the Point Grey Peninsula.

The pumped flow would be 2 x ADWF (North Shore) + 1 x ADWF (8th Avenue) = 280 ML/d + 428 ML/d = 708 ML/d. As for Option 4, a single pumping station has been considered at this stage.

7.6 ALTERNATE NORTH SHORE SITE

The intent of considering an alternate North Shore site is that all of the existing Lions Gate facility would become abandoned (except for the influent pumping station and the outfall).

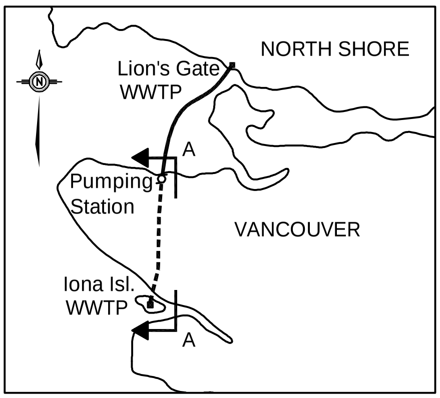
The existing Lions Gate influent pumping station has been assumed to be suitable to convey West Vancouver flows and a new pumping station is considered to be required for North Vancouver flows.

A schematic of a proposed alternate wet weather treatment facility is shown as Figure 8.

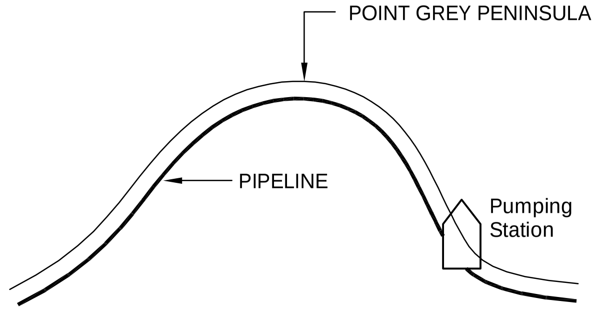
All flows would receive screening grit separating and odour control. Flows up to 2 x ADWF would then be pumped through a new pumping station to the Iona WWTP. Flows in excess of 2 x ADWF would receive primary treatment plus disinfection.

The effluent would then be pumped back to Lions Gate for discharge through the existing outfall. Sludge from primary treatment would be pumped into the suction side of the Iona Pumping Station.

Odour control would be included for most process components as detailed on Figure 8.

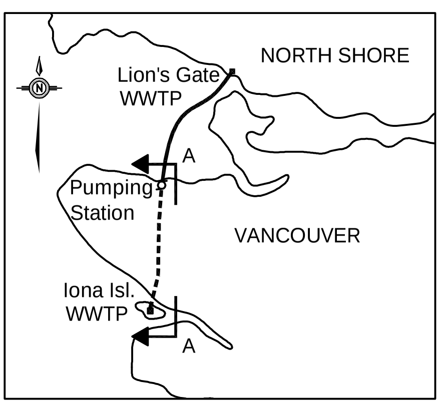


OPTION 4

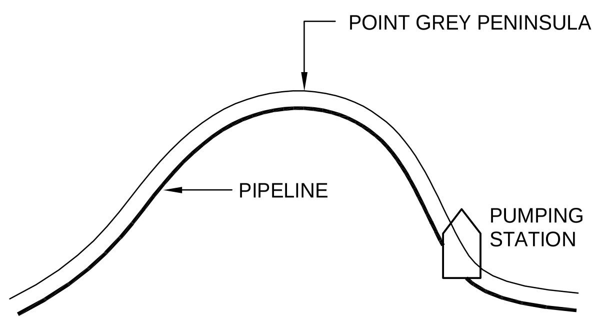


SECTION A-A

- ENGLISH BAY ALIGNMENT OF TWO 965mm Ø PIPELINES
- PLUS SINGLE 1270mm Ø FORCEMAIN OVER HIGHBURY AREA



OPTION 5



SECTION A-A

- ENGLISH BAY ALIGNMENT OF TWO 965mm Ø PIPELINES
- PLUS SINGLE 1800mm Ø FORCEMAIN OVER HIGHBURY AREA

POSSIBLE FORCEMAIN ROUTES
REFINED OPTIONS 4 AND 5

FIGURE 6

ACAD DWG. FIGURE 5_6 1:1 04-01-13

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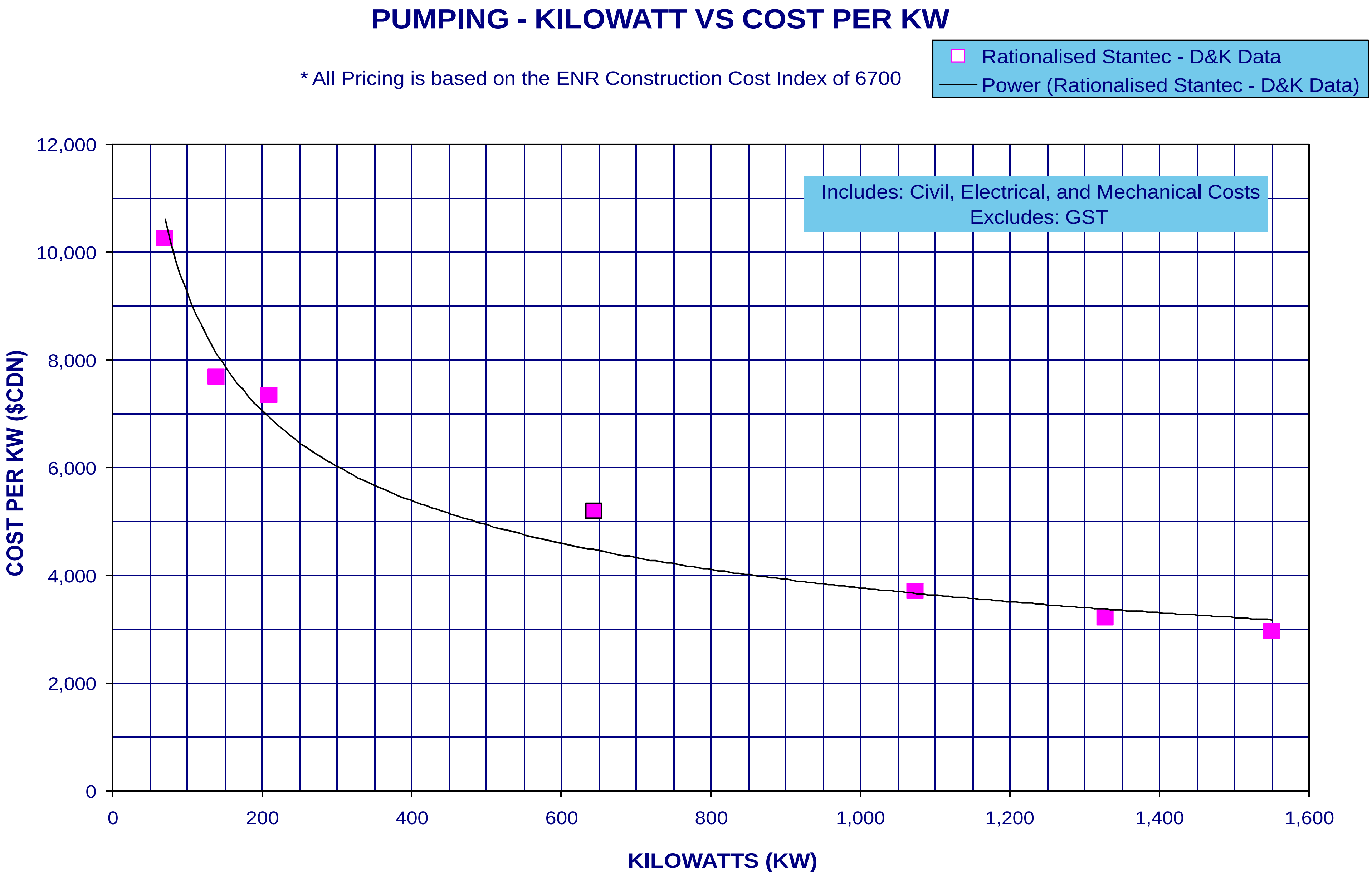
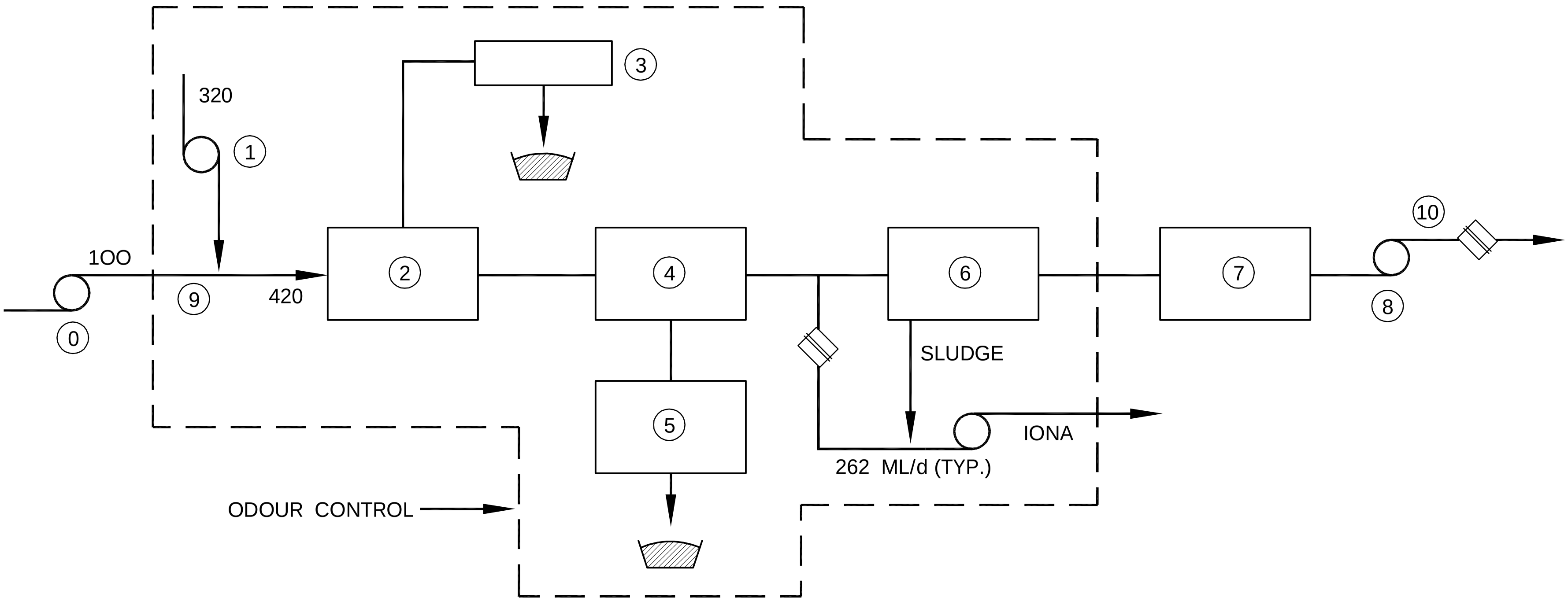

PUMPING - KILOWATT VS COST PER KW
* All Pricing is based on the ENR Construction Cost Index of 6700
Rationalised Stantec - D&K Data
Power (Rationalised Stantec - D&K Data)
Includes: Civil, Electrical, and Mechanical Costs
Excludes: GST
COST PER KW ($CDN)
12,000
10,000
8,000
6,000
4,000
2,000
0
0
200
400
600
800
1,000
1,200
1,400
1,600
KILOWATTS (KW)

Figure 7



MECHANICAL

2 SCREENS
3 COMPACTOR
4 GRIT SEPARATOR
5 WASHER

HIGH RATE TREATMENT

6 PRIMARY SEDIMENTATION
7 CHLORINE CONTACT

PUMP STATIONS

0 WEST VANCOUVER FLOWS
1 NORTH VANCOUVER FLOWS
8 RELOCATED WWTP SITE DISCHARGE to LIONS' GATE WWTP

PIPELINES

9 LG to RELOCATED WWTP SITE
10 RELOCATED WWTP SITE to EXISTING OUTFALL at LION'S GATE SITE

ACAD DWG. FIGURE 8 04-06-23

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SCHEMATIC OF TREATMENT AND PUMPING FOR A RELOCATED WWTP SITE

FIGURE 8

8 COST ESTIMATES (INCLUDING O&M COSTS)

A table of Concept Cost Estimates and options were initially prepared (Table 2). These estimates and options were subsequently refined and the results are included in Table 4. Background data related to Table 4 are included in Table 3. Both these tables summarize the cost estimate data for the five options considered.

Comments related to the estimates in Table 4 follow.

For the pump stations, and based on our assessment from other projects, the adopted split in capital costs is as follows:

- Civil – 50%
- Mechanical – 30%
- Electrical – 20%

In assessing the operating energy costs a curve was developed by Stantec and Dayton & Knight Ltd. A copy is enclosed as Figure 7.

The electrical demands have been based on a flow equivalent to 2 x ADWF and the annual energy required has been based on 1.2 x ADWF.

From previous Dayton & Knight Ltd. experience we have used a value of 0.5% for annual maintenance when compared with the capital cost for pipelines and civil works and 5% for mechanical and electrical. These percentages have been adopted for this study.

In calculating Net Present Values, a 6% discount factor has been used, as directed by GVRD.

Because expansion work at Lions Gate is to occur by 2031, we have assumed a 3 year construction period with capital cost requirements starting in 2028 (see Table 3).

For the North Shore Diversion option, it has been assumed that flows greater than 2 x ADWF would be pumped and treated at a new facility located near the existing treatment plant site. The costs associated with that facility are also shown on Table 4.

TABLE 3
NORTH SHORE DIVERSION COST ANALYSIS

Description	Factor/ Input	Option 1 LG to Highbury 1250mm	Option 2 LG to Hibury + 3 m tunnel	Option 3 LG to Hibury + 6m Tunnel	Option 4 Option 1+ Force Main 1250mm	Option 5 Similar to 4 with ADWF from 8t St. Interceptor
Capital Cost $		$81,253,000	$156,253,000	$181,253,000	$111,753,000	$144,753,000
Pipelines and/or Tunnel		$28,000,000	$103,000,000	$128,000,000	$38,500,000	$43,500,000
Wet weather flow plant on relocated site		$43,253,000	$43,253,000	$43,253,000	$43,253,000	$43,253,000
Pump Stations		$10,000,000	$10,000,000	$10,000,000	$30,000,000	$58,000,000
Division1 Cost	2.50%	$2,031,325	$3,906,325	$4,531,325	$2,793,825	$3,618,825
Engineering	16.00%	$13,000,480	$25,000,480	$29,000,480	$17,880,480	$23,160,480
Project Management / Quality Control	4.00%	$3,250,120	$6,250,120	$7,250,120	$4,470,120	$5,790,120
Contingency	30.00%	$24,375,900	$46,875,900	$54,375,900	$33,525,900	$43,425,900
Total Capital Cost		$123,910,825	$238,285,825	$276,410,825	$170,423,325	$220,748,325
Pump Stations and Move to Relocated Site	100.00%	$53,253,000	$53,253,000	$53,253,000	$73,253,000	$101,253,000
Civil	50.00%	$26,626,500	$26,626,500	$26,626,500	$36,626,500	$50,626,500
Mechanical	30.00%	$15,975,900	$15,975,900	$15,975,900	$21,975,900	$30,375,900
Electrical	20.00%	$10,650,600	$10,650,600	$10,650,600	$14,650,600	$20,250,600
Maintenance Cost $/annum						
Pipelines and/or Tunnel % of Capital	0.25%	$70,000	$257,500	$320,000	$96,250	$108,750
Civil % of Capital	0.25%	$66,566	$66,566	$66,566	$91,566	$126,566
Mechanical % of Capital	2.00%	$319,518	$319,518	$319,518	$439,518	$607,518
Electrical % of Capital	2.00%	$213,012	$213,012	$213,012	$293,012	$405,012
Total Maintenance		$669,096	$856,596	$919,096	$920,346	$1,247,846
Energy (based on Flow)						
Energy at year (Max) ($/annum)	2081	$380,000	$380,000	$380,000	$1,580,000	$4,408,000
Total Dynamic Head m		10.00	10.00	10.00	120.00	130.00
Static Head m		5.00	5.00	5.00	91.00	96.40
Exponent of Friction Loss	2.00					
Economic Parameters						
Discount Rate	6.00%					
Commissioning	2031					
End Evaluation Year	2061					
Results		**Cost $**	**Cost $**	**Cost $**	**Cost $**	**Cost $**
Total NPV		**$30,138,000**	**$55,849,000**	**$64,419,000**	**$44,404,000**	**$64,394,000**
NPV	**Year 2003**	O&M $2,870,183 Capital $27,267,676	O&M $3,411,767 Capital $52,436,909	O&M $3,592,294 Capital $60,826,654	O&M $6,900,421 Capital $37,503,164	O&M $15,815,698 Capital $48,577,627

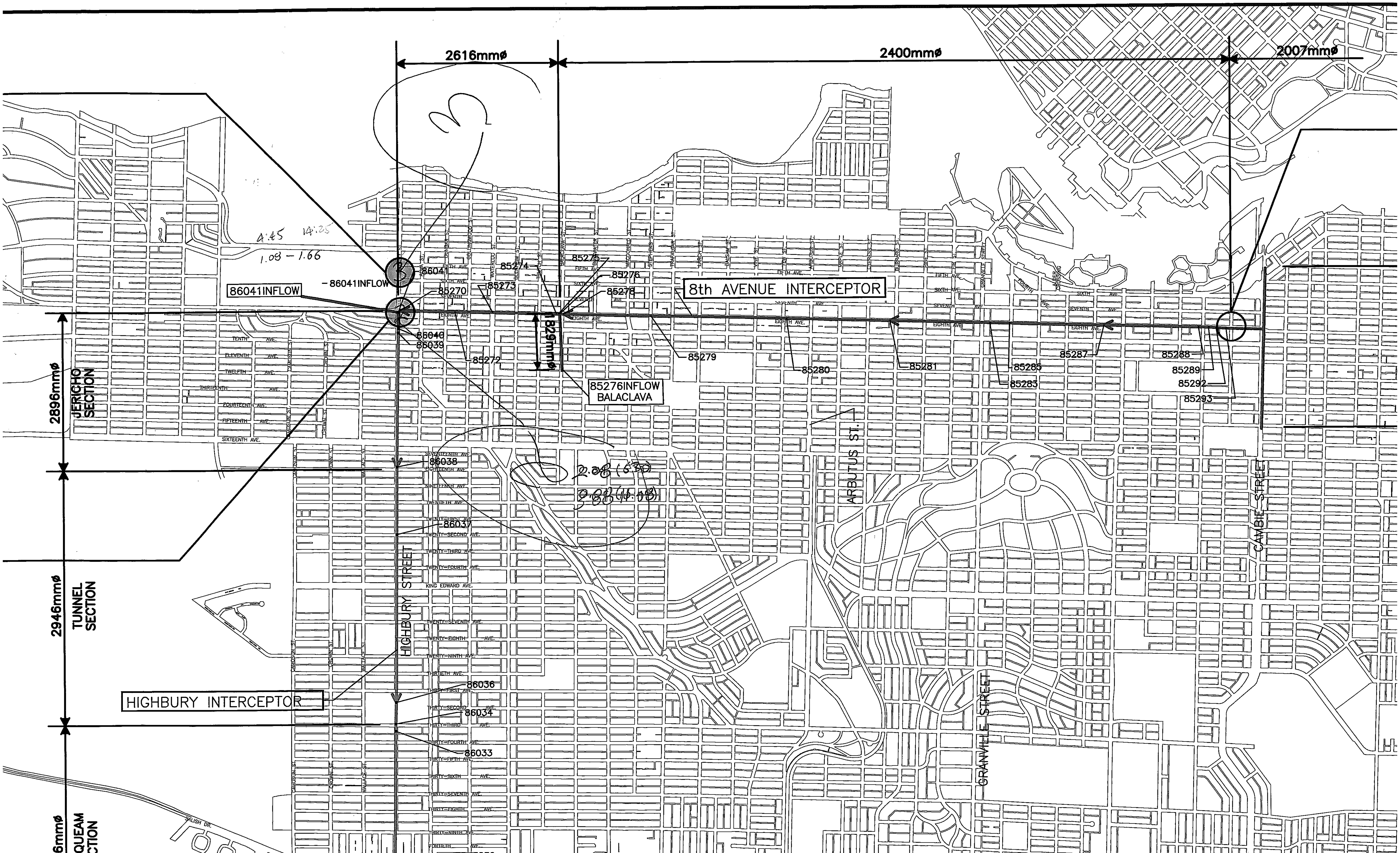
TABLE 4
NORTH SHORE DIVERSION TO IONA ISLAND WWTP COST ESTIMATES OF SELECTED OPTIONS
(INCLUDING O&M COSTS)

Option	Description	Concept Costs in Millions of Dollars								
		Pumping Station	Moving to Alt. Site	Marine Pipeline	Highbury Tunnel	Shallow Forcemain System (incl. P.S.)	Total Capital	Present Worth		
								Capital	O&M (2046*)	Total
1	Pumping to Highbury	15.3	66.0	42.7	---	---	124	27.3	2.8	30.1
2	As 1) with parallel 3 m tunnel	15.3	66.0	42.7	114.4	---	238	52.4	3.4	55.8
3	As 1) with 6 m tunnel	15.3	66.0	42.7	152.5	--	276	60.8	3.6	64.4
4	As 1) with shallow bury forcemain above Highbury system for 2 x ADWF	45.8	66.0	42.7	---	16.0	170	37.5	6.9	44.4
5	Similar to 4) plus ADWF from 8th Ave. Interceptor	88.5	66.0	42.7	---	23.6	221	48.6	15.8	64.4

Capital cost include the following:

- Division 1 2.5%
- Engineering 16%
- Project Management/Quality Control 4%
- Contingency 30%
- Discount Rate 6% pa

APPENDIX A – 8TH AVENUE DATA


2616mmø
2400mmø
2007mmø
1829mmø
2896mmø
JERICHO SECTION
2946mmø
TUNNEL SECTION
QUEAM SECTION
86041INFLOW
- 86041INFLOW
86041
85270
85273
85274
85275
85276
85278
86040
86039
85272
8th AVENUE INTERCEPTOR
85279
85280
85281
85285
85283
85287
85288
85289
85292
85293
85276INFLOW
BALACLAVA
86038
86037
86036
86034
86033
HIGHBURY INTERCEPTOR
HIGHBURY STREET
ARBUTUS ST.
GRANVILLE STREET
CAMBIE STREET
KING EDWARD AVE.
EIGHTH AVE.
TENTH AVE.
ELEVENTH AVE.
TWELFTH AVE.
THIRTEENTH AVE.
FOURTEENTH AVE.
FIFTEENTH AVE.
SIXTEENTH AVE.
SALISH DR.
4.45 14.25
1.08 - 1.66

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 7
Interim Solids Handling Facilities

FINAL REPORT

Prepared for

Greater Vancouver Regional District

Prepared by

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Dayton & Knight Ltd.
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North Vancouver, BC
V7P 3S1

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(Stantec Consulting Ltd.)
Tel: (604) 597-0422
Fax: (604) 591-1856

August 2005

117-00018

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1 INTRODUCTION

Solids materials and biosolids produced in the wastewater treatment processes require processing, and arrangements for final disposal and/or recycling. Sludge management in the GVRD has always been a challenge due to the following factors:

- Evaluate the existing facilities and operational conditions
- Increasing sewage flow and loads in the region.
- Regulatory requirements (BC Organic Matter Recycling Regulation, OMRR) and public/industry acceptance.
- Uncertainty about sludge recycling options (ranch land application, gravel pits, mining sites silviculture etc.).
- Uncertainty about biosolids market needs.
- More sludge production due to future treatment process upgrade (from primary to secondary).

Primary sludge and grit/screenings are currently produced at Iona Island Wastewater Treatment Plant (IIWWTP) and Lions Gate Wastewater Treatment Plant (LGWWTP). With the introduction of biological or chemical units for treatment upgrades in interim and build-out to secondary stages, substantial sludge quantity increases are expected at both plants. A proper sludge management plan is important to ensure the capacity and capability for sludge handling.

The Greater Vancouver Regional District (GVRD) Nutrifor Program (formerly Biosolids Recycling Program) has initiated efforts to develop a regional sludge management plan and identify potential sludge recycling options (GVRD Biosolids Recycling Program Annual Report Draft, 2003). The objectives of this appendix are to investigate the potential increases in sludge production resulting from the interim and build-out to secondary upgrades, and to recommend interim sludge handling options for IIWWTP and LGWWTP. The scope of work in this study includes the following:

- Assess current sludge handling unit processes, including sludge thickening, sludge stabilization, and dewatering.
- Predict sludge quality and quantity of interim and build-out to secondary process upgrades identified in Appendix 3 and Appendix 4, including the side stream (sludge supernatant recycles) impacts to the treatment plants.
- Identify potential treatment technologies to reduce sludge volume and improve sludge quality.
- Recommend interim treatment option to handle sludge and provide operating flexibility to meet regulatory and market requirements.

During the initial project meeting with GVRD staff and subsequent discussions at the workshops, the following scenarios and conditions are suggested to best utilize the existing facilities and accommodate the future needs:

- Continue to use and expand (if needed) mesophilic anaerobic digestion for sludge stabilization at IIWWTP during interim operation, with the capability to be converted to thermophilic anaerobic mode when the plant is upgraded to provide secondary treatment by 2021. Continue to use lagoons for land drying to their full capacity for as long as practical.
- Continue to use and expand (if needed) thermophilic anaerobic digestion for sludge stabilization and centrifuge dewatering at LGWWTP during interim operation.
- Provide flexibility to produce different qualities of biosolids end product subject to the regulatory requirement for different land application (e.g. Class A or Class B Biosolids, for non-restricted or restricted use).
- Identify potential technologies to maximize the current and interim sludge processing capacity and treatment.
- Examine a range of alternate stabilization/processing technologies which might be employed by GVRD in the future and advise on how appropriate they might be for future use.

2 EXISTING SLUDGE HANDLING FACILITIES DESCRIPTION

2.1 IONA ISLAND

Iona Island WWTP is currently equipped with gravity thickeners, mesophilic anaerobic digesters, storage/settling lagoons, and land drying for its sludge processing.

2.1.1 Current Facility Capacity

Gravity Thickener

In 2002, the annual averages of primary sludge withdrawn from the primary sedimentation tanks were about 41 t/d (dry solids) and 9.8 MLD. The thickened sludge concentration averaged about 5.7%. Normally with one thickener in service and the other unit standby, the operation achieves approximately 97% of solids capture at current loading condition. The volume reduction rate was about 10:1 (thickener influent vs. thickener underflow to digester).

The design solids and hydraulic loadings of the gravity thickener are 45 $kg/m^2/d$ and 32 $m^3/m^2/d$, respectively (GVRD 2000). Current solids loading is estimated about 67 $kg/m^2/d$, which has been about 50% higher than the plant design loading rate. However, compared to a typical design rate (87 ~ 136 $kg/m^2/d$ for primary sludge only) and current capture efficiency, these two gravity thickeners are capable of handling higher solids loading than the original design rate. Typical hydraulic loading rate of gravity thickener is about 24 ~ 30 $m^3/m^2/d$. The average hydraulic loading with one unit in service was about 19 $m^3/m^2/d$ (2002 annual average), which was only about 60% of the plant design rate (32 $m^3/m^2/d$).

As shown in Figure 2.1, the solids and hydraulic capacities with two thickener units in service are estimated about 62 t/d (100 $kg/m^2/d$) and 18.5 MLD (30 $m^3/m^2/d$), respectively. The solids loading rather than the hydraulic loading rate usually become the limiting factor of the gravity thickener capacity. When the loading rates increase, the solids capture rate will be compromised and the thickened sludge concentration will be reduced (e.g. less than 5%). When the solid loading rate exceeds the system capacity, lower solids capture efficiency and higher solids concentration in the thickener overflow (supernatant) are expected.

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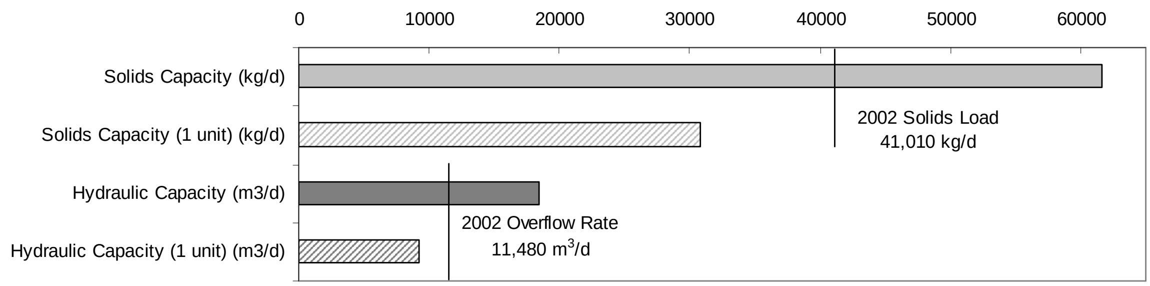
FIGURE 2.1
IIWWTP GRAVITY THICKENER – SOLIDS AND HYDRAULIC CAPACITIES



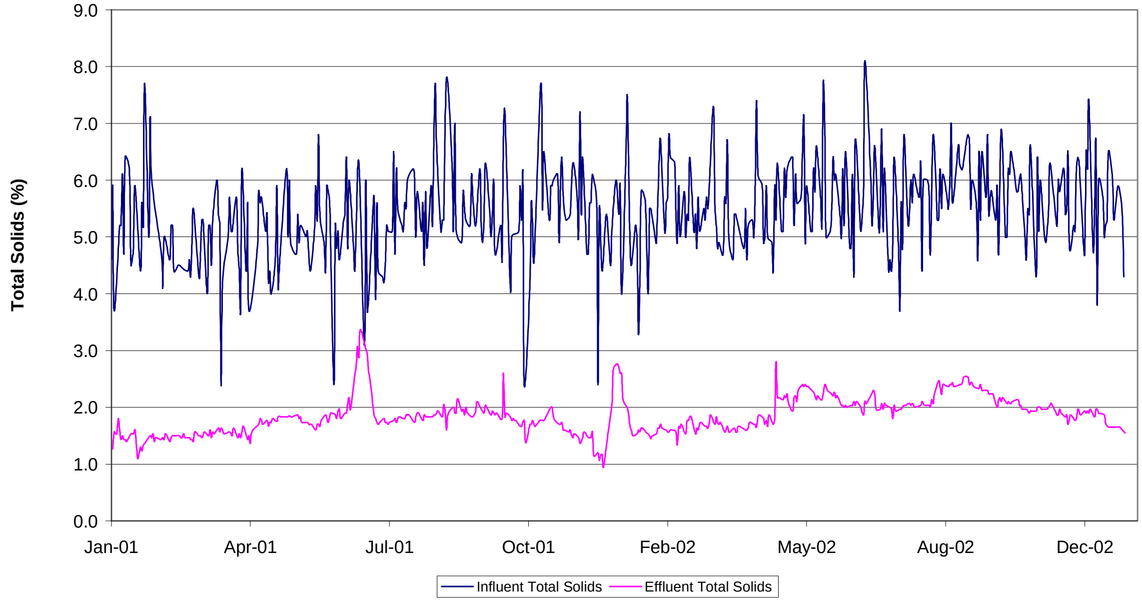
The gravity thickener performance can be improved by lowering the withdraw rate and increasing the pump operating time, to prevent instantaneous shock loadings due to the intermittent pumping from primary sedimentation underflow. Continuous sludge withdraw from the primary sedimentation is a better operation to optimize the gravity thickener performance. To assure the efficiency of digester performance, thickener capacity must be upgraded to handle more primary sludge production when the flow and loads increase, to achieve minimum thickened sludge concentration at 5%.

Gravity thickener is most effective for primary sludge and chemically enhanced primary sludge (CEP sludge). Additional thickener units will be required to handle more primary or CEP sludge production when the flow and loads increase. Other sludge thickening processes such as dissolved air flotation (DAF) are considered efficient for biological sludge thickening (e.g. biosolids generated from CAS, HRAS and TF/SC options for interim and build out to secondary upgrades).

Anaerobic Digester

The thickened primary sludge is pumped to the anaerobic digester for sludge stabilization. The average digester influent and effluent total solids sludge concentration, shown in Figure 2.2, are about 5.4% and 1.8 %, respectively. In 2002, the thickened sludge averaged about 40 t/d (dry solids) and 910 m^3/d (wet volume). The digester solids loading and hydraulic retention time are estimated about 2.2 kg VS/m^3/d and 16 days, respectively, with three (3) digester units in service on average. The operating temperatures ranged between 38 ~ 40 ºC, which were at the high end of mesophilic condition (typical 30 ~ 38 ºC). Typical design capacity of single-stage complete-mix high rate mesophilic anaerobic digester is operated between 2.4 ~ 4.3 kg VS/m^3/d, with a HRT of about 10 ~ 20 days. The original design capacity with four digester units is 3.2 kg VS/m^3/d and 20 days of HRT.

FIGURE 2.2
IIWWTP DIGESTERS – AVERAGE SLUDGE CONCENTRATIONS (2001~2002)



The digester operation has been varied over the years and several system upsets have been experienced in some units. Single-stage and two-stage operations have been used alternately in the past years. Currently, digesters #1 and #3 are operated as the primary digesters, followed by digester #2 and #4 as the secondary, respectively. This two-stage digestion operation is expected to prevent short-circuiting, enhance VS destruction rate, and provide system stability.

An average of 73% volatile solids (VS) destruction was achieved in 2000. Complete VS destruction data of 2001~2003 were not available during this evaluation. In 2004, two-stage operation with digester #3 and #4 in series averaged 75% VS destruction (January 2004 ~ November 2004). During the same period, single-stage operation with digester #2 only (digester #1 was offline) achieved 62% VS destruction. Two-stage operation in 2004 achieved pathogen reduction of ***Faecal coliforms*** less than 200,000 MPN/g dry solids (digester #3 and #4 in series). Single-stage operation in 2004 (digester #2 only) often exceeded the BC Organic Matter Recycling regulation (OMRR) Class B biosolids minimum requirement of less than 2,000,000 MPN/g dry solids.

The digester capacity is usually limited by the hydraulic retention time (HRT) requirement. At a minimum of 20 days HRT, the digester capacity is estimated about 990 m^3/d (with 4 digesters in service). When one of the digester is out of service for maintenance or repairing (e.g. digester #1), the system capacity is reduced to about 740 m^3/d, which is considered insufficient for even current minimum demand. The VS destruction efficiency will be compromised when the operating HRT is reduced, for example, in the case of single-stage operation (digester #2 only) during 2004.

There is no immediate concern about the digested sludge quality and efficiency, because land drying and on-site stockpiling are the current dewatering and disposal arrangement. However, some lagoon and stockpiling space will no longer be available in the near future (e.g. by 2006~2007) due to interim upgrade and site preparation requirements. It will become an operating concern with existing digester capacity to produce good quality biosolids (e.g. Class A or Class B biosolids) for recycling purposes to meet the BC Organic Matter Recycling Regulation and GVRD Biosolids Recycling Program standards, particularly when one or two digesters are offline for O/M purposes. The biosolids reuse/recycling options will be limited by the deterioration of biosolids quality. Expansion of the digester capacity and system upgrade to thermophilic operation should be considered to accommodate future needs.

Sludge Storage/ Settling Lagoons followed by Land Drying

The digested sludge is further stabilized and thickened in four (4) settling lagoon cells adjacent to the plant. The lagoon cells have a surface area of 115,000 m^2 at 3.3 m side water depth (SWD) and a total volume of 334,000 m^3. About 910 m^3/d of digested sludge (2002 average) was produced from the digesters and discharged in these storage/settling lagoons. Currently, the digested sludge is pumped to one of the two cells alternately, and the supernatant is recycled to the plant for treatment. The other two cells are full and will not receive further sludge discharge before dredging.

Annual sludge dredging is scheduled to remove 50% of the volume from one cell only. The average solids retention time in the drying lagoon is estimated about 8 years based on current dredging schedule. The dredged sludge has a solids content of 20 to 30% and is stockpiled on the adjacent land east of the treatment plant for drying. In 2002, only about 25% of the stockpile sludge in wet volume (approximate 70% solids concentration after drying) was hauled offsite for land application or other reuse. Current sludge inventory on IIWWTP site was estimated about 524,000 m^3 in wet volume (370,000 m^3 in lagoons and 150,000 m^3 in stockpile).

Based on a solids loading rate of 0.25 kg VS/m^2·d (Metcalf and Eddy, 2003), the existing lagoons have approximately 30% remaining capacity. A GVRD study also estimated the remaining lagoon storage capacity is about 25% (GVRD 2000). The remaining lagoon capacity is sufficient until 2021 if no interim upgrade is carried out (primary sludge only). The lagoon solids loading will reach 0.25 kg VS/m^2/d within 4~8 years with various interim upgrade options. As the solids loading increases to reach 0.25 kg/VS/m^2/day, the efficiencies of dewatering, of additional VS destruction and of pathogen kill will be degraded.

As a result of site preparation and construction for both the interim upgrade and build-out to secondary some of existing land used for stockpiling and one of the lagoon cell will no longer be available. Potential upgrade options for sludge drying/dewatering are further discussed in Section 6.

2.1.2 Sludge Quality and Quantity

The average sludge quality and quantity for 2002 are summarized in Table 2.1, for thickened sludge, digested sludge, lagoon dewatered sludge and stockpile sludge, respectively. The thickener solids capture efficiency is estimated about 97%, and the digester volatile solids (VS) destruction rate was estimated about 71%.

TABLE 2.1
IIWWTP SLUDGE QUANTITY AND QUANTITY

Sludge Type	Unit	2002 Average	Range
Thickened Sludge			
Flow Rate	m^3/d	910	-
VS/TS Ratio	-	0.87	0.65~0.9
Solids Concentration	%	5.7	3.3~8.1
Digested Sludge			
Flow Rate	m^3/d	910	-
VS/TS Ratio	-	0.64	0.54~0.77
Solids Concentration	%	2.0	1.2~2.6
Lagoon Sludge			
Flow Rate/Volume	m^3/yr	22,300*	-
VS/TS Ratio	-	0.45*	-
Solids Concentration	%	20*	-
Stockpile Sludge			
Flow Rate/Volume	m^3/yr	4500*	-
VS/TS Ratio	-	0.20*	-
Solids Concentration	%	70*	-

*: Estimates

The values of nutrient contents in sludge/biosolids are available in GVRD's monthly composite sample database, including micronutrients (trace metals and organic compounds) and macronutrients (N, P, potassium, calcium and magnesium). Many trace metals present in sludge can be beneficial as fertilizer or soil conditioner. However, excessive metals may become a concern regarding the BC OMRR standards, particularly the concentrations of mercury, zinc, and cadmium.

The digested primary sludge typically consists of 3% nitrogen and 2% phosphorus, respectively. However, the values of these macronutrients for land application are subject to their available format in soil. The available nutrient contents are generally reduced after a long retention time (e.g. 8 years) in the storage lagoons. GVRD has planned to conduct a detailed survey in 2004 at IIWWTP to determine the dewatered sludge characteristics.

2.1.3 Non-Recyclable Residuals

The non-recyclable residuals generated from treatment plant operation include the screenings and grit. Screenings captured by the 12.7 mm opening mechanical screens are conveyed to the compactors for dewatering. A rotary drum screen with 9 mm perforated opening is operated to capture screenings in the non-domestic TLW. The screening rejects are washed and pressed by hydraulic rams to reduce the volume, then collected in a disposal bin. The dewatered screenings are processed in the Waste-to-Energy Facility located in Burnaby, BC (2000 ~ mid 2004).

Grit collected from the bottom of grit chambers and pre-aeration tanks are pumped to one of the two grit cyclones and classifiers where the grit is concentrated and washed. The grit collected in the digester (annual cleaning alternating among four digesters) and non-domestic trucked liquid waste (TLW) pre-treatment is processed by the same grit dewatering facilities. The washed grit is transported to landfills for disposal.

The Contracted Services Division of GVRD Engineering and Construction Department currently administers the hauling and disposal of WWTP residuals. The hauling records (per hauling trip) of the screenings and grit are shown in Figure 2.3 and 2.4, respectively. The annual residual wet volumes are summarized in Figure 2.5 (2001~2004). In 2002, total screening and grit productions are about 270 and 1,600 wet tonnes/year, respectively. The total expenditures of residual hauling were $123,000 plus GST in 2001 and $110,000 plus GST in 2002, respectively.

FIGURE 2.3
IIWWTP BAR SCREENINGS PRODUCTION (2001 – 2002)

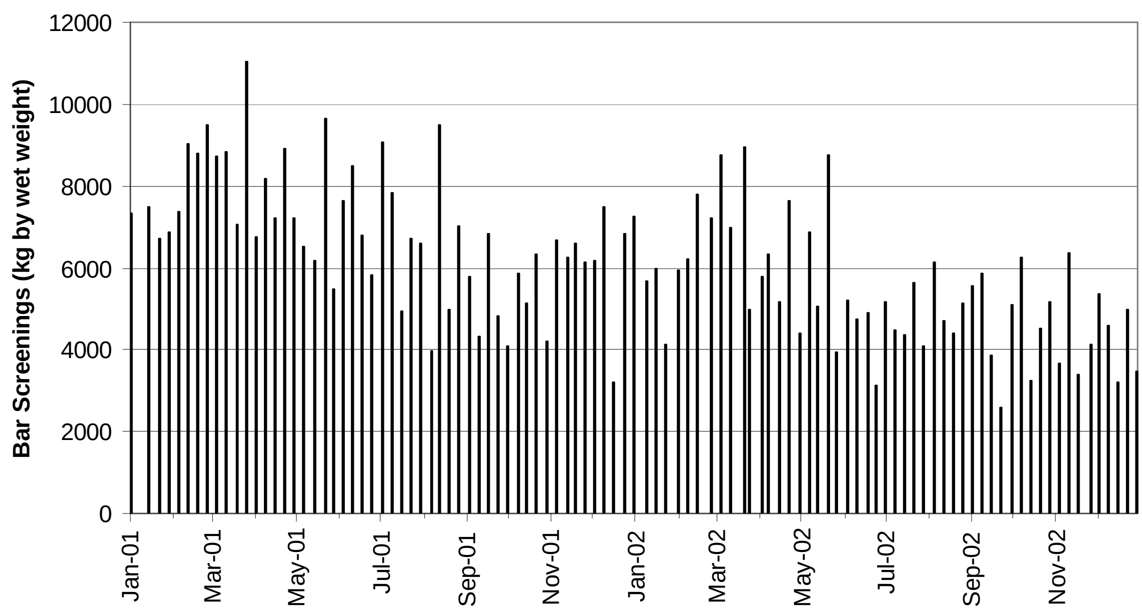


FIGURE 2.4
IIWWTP GRIT PRODUCTION (2001 – 2002)

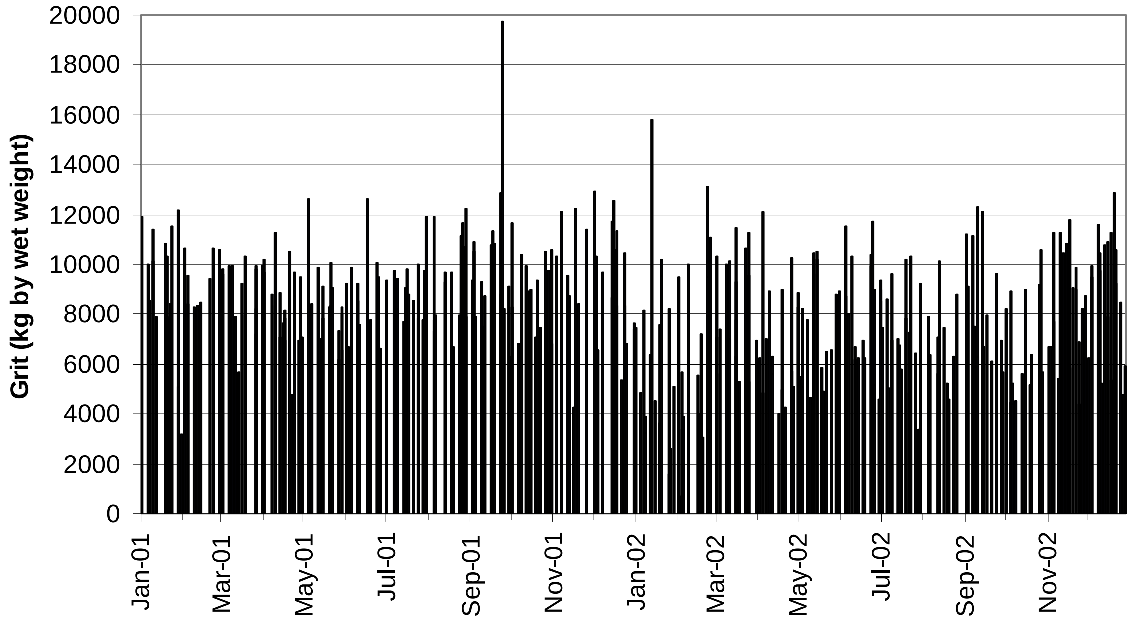
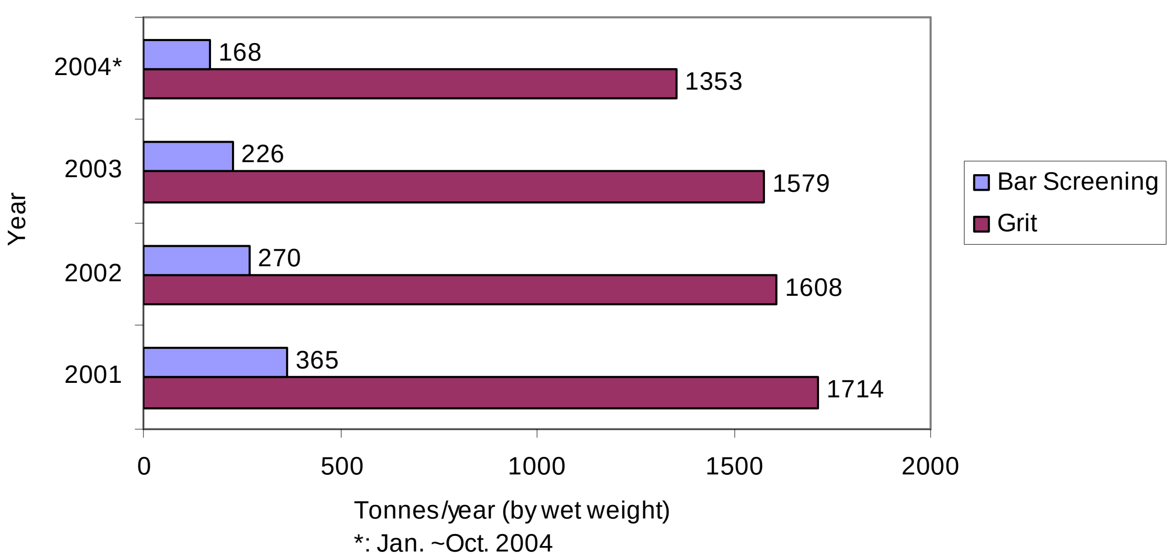


FIGURE 2.5
IIWWTP ANNUAL NON-RECYCLABLE RESIDUALS (2001 – 2004)



The characteristics of non-recyclable residuals, as well as their leachate samples, were investigated in a GVRD study (Dayton & Knight, 1999). Results suggested that their leachate metals concentrations were well below the standards specified in the BC Special Waste Regulation. Landfilling is still the usual method for grit disposal, and screenings are best incinerated with municipal solid waste for energy recovery. The non-recyclable residuals quantity will increase when the flow and loads increase. Also the removal efficiency improvements of the screens and grit chambers will increase quantities of solids.

The deficiencies of grit removal efficiency were identified in the Grit Removal Study and System Upgrade Predesign Report (Dayton & Knight, 1999), particularly during wet weather conditions. Carryover of fine grit and re-suspension has resulted in grit deposit in the sludge digesters. The expansion of the grit removal and dewatering capacities were recommended to improve the grit capture efficiency and reduce the grit volume. Redundancy capacity can be provided in such expansion to allow for service downtime. However, such expansion should be considered in accordance with headworks, flow split, and major process upgrades during interim and build-out stages.

Screenings/grit productions of 2003 and 2004 at IIWWTP have gradually decreased to 168 wet tonnes/year and 1,353 wet tonnes/year, respectively (see Figure 2.5). Such reductions could be caused by the efforts of sewer separation in the catchments and/or reduced removal efficiency due to flow and load increases. In comparisons with Annacis Island and Lulu Island WWTPs, IIWWTP produced less screenings but more grit on tonnes per ML sewage basis, however such differences are subject to sewer collection system (combined or separate sewer system) and unit process efficiencies (bar screens, grit chambers and grit washer/dewatering):

Average non-recyclable residual productions (annual wet weight) of 2001 ~ 2004

- IIWWTP: screenings (1.3 tonnes/ML), grit (7.3 tonnes/ML)
- AIWWTP: screenings (3.4 tonnes/ML), grit (3.7 tonnes/ML)
- LIWWTP: screenings (6.8 tonnes/ML), grit (5.8 tonnes/ML)

2.2 LIONS GATE

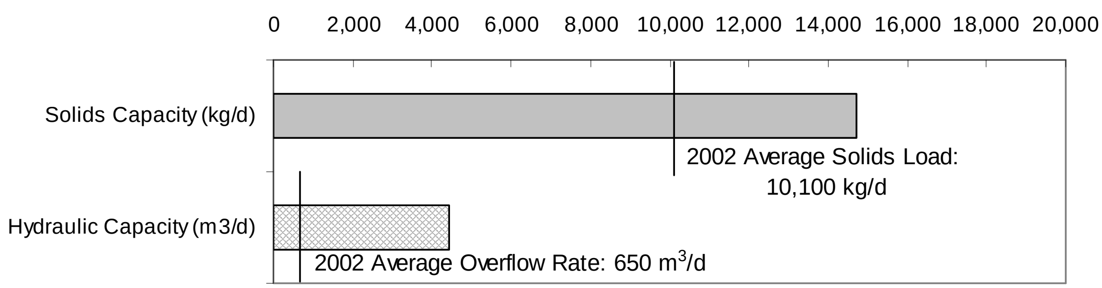
Lions Gate WWTP is currently equipped with gravity thickeners, thermophilic anaerobic digesters, and centrifuge dewatering for its sludge processing. The dewatered sludge is transported to offsite locations for land application.

2.2.1 Current Facility Capacity

Gravity Thickener

There is one 13.7m diameter circular gravity thickener in LGWWTP to handle the primary sludge removed from the primary sedimentation tanks. The original design solids and hydraulic capacities of the gravity thickener at LGWWTP are 100 $kg/m^2/d$ and 30 $m^3/m^2/d$, respectively (GVRD 2001), which are within the typical design ranges. The existing sludge thickener capacities are projected in Figure 2.6, at approximately 15 tonnes/d of solids load and 4.4 MLD of hydraulic overflow load. The annual average loading rates of 2002 were about 10 tonnes/d of solids and 0.65 MLD of sludge flow. Since there is only one (1) gravity thickener at LGWWTP, there is no redundancy capacity available for maintenance need.

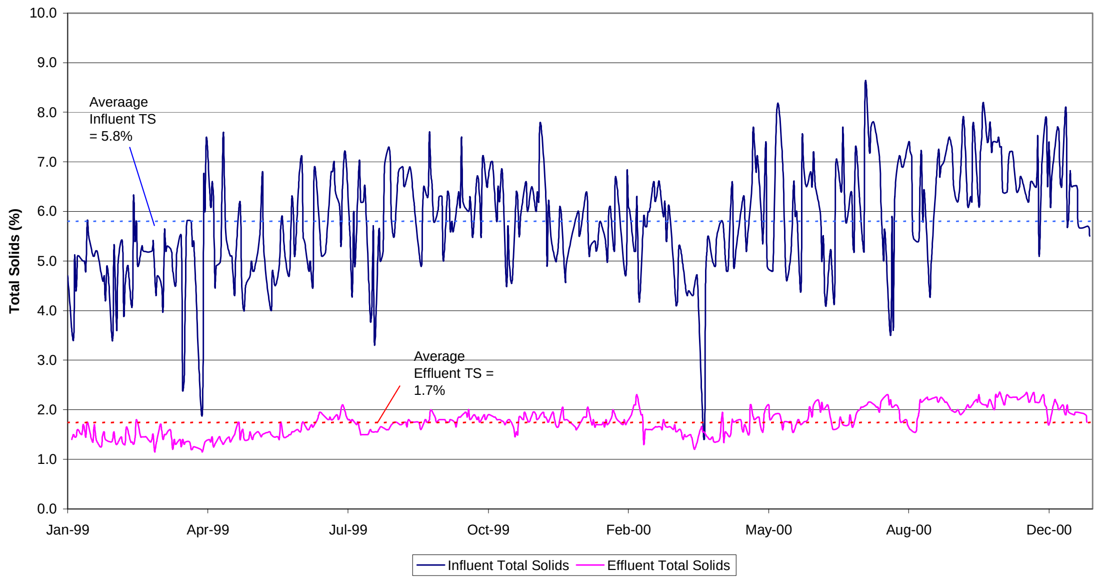
FIGURE 2.6
LGWWTP GRAVITY THICKENER – SOLIDS AND HYDRAULIC CAPACITIES



Anaerobic Digester

Two 3,100 m^3 anaerobic digesters (digester #4 as primary and #3 as secondary) are operated in series to achieve sludge stabilization. Old digesters #2 is no longer in use and Digester #1 is operated as a digested sludge storage tank (DSST). A major concern is that if this tank is not available there will be no back-up since the design of digester 3 does not allow for fill and draw operation. Thickened sludge from the sludge thickener and thickened scum from the scum thickener are stabilized in the digesters and undergo thermophilic digestion at approximately 55ºC. The digestion system is operated as an extended thermophilic condition to achieve 22 days solids retention time (SRT) at 55ºC. With current (year 2000) sludge loads of 198 m^3/day and 11,978 kg TS/day, the calculated average SRT is 15 days with one digester in operation, and 31 days with two digesters in service. Digester sludge is recirculated throughout the tank by recirculation pumps and gas mixing systems. The average digester influent and effluent solids sludge concentration shown in Figure 2.7 are about 5.8% and 1.7%, respectively (1999~2000).

FIGURE 2.7
LGWWTP DIGESTERS – AVERAGE SLUDGE CONCENTRATIONS (1999 – 2000)



Centrifuge Dewatering

Two Alfa-Laval Sharples Centrifuges dewater digested sludge from 2~2.5% TS to 30 ~ 35% TS. Digested sludge is batch pumped from digester No. 3 to either one of the centrifuges. The sludge passes through an inclined macerator, dosed with polymer then fed to the centrifuge. Dewatered biosolids are conveyed into trucks and hauled offsite for disposal. Centrate flows by gravity back into the influent distribution channel.

The rated centrifuge capacity is about 80 ~ 100 L/min per unit, which is just sufficient to handle current sludge volume based on 16 hour per day operation.

2.2.2 Sludge Quality and Quantity

The average sludge quality and quantity for 2000 are summarized in Table 2.2, for thickened sludge, digested sludge, and centrifuged sludge, respectively. The thickener solids capture efficiency is estimated about 93%, and the digester volatile solids (VS) destruction rate averages about 78%. The centrifuged sludge wet volume is approximated by an average of 35% dewatered sludge concentration.

TABLE 2.2
LGWWTP SLUDGE QUALITY AND QUANTITY (2000)

Sludge Type	Unit	2000 Average	Range
Thickened Sludge			
Flow Rate	m^3/d	198	91 - 337
VS/TS Ratio	-	0.88	0.73 - 0.92
Solids Concentration	%	6.1	1.4 - 8.6
Digested Sludge			
Flow Rate	m^3/d	198	91 - 337
VS/TS Ratio*	-	0.69	0.61 - 0.76
Solids Concentration*	%	1.9	1.2 - 2.4
Dewatered Sludge			
Flow Rate	m^3/d	10	-
VS/TS Ratio	-	0.65	-
Solids Concentration	%	35	-

*Average of digester No. 3 and No. 4

Current treatment efficiencies of the sludge handling units (gravity thickeners, digesters and centrifuges) are considered satisfactory. However, there is no extra gravity thickener unit to provide redundancy for maintenance and repairing needs. The extended mode of thermophilic anaerobic digestion (22 days of SRT at 55ºC) is capable of achieving high degree of solids reduction and pathogen kills, however, high heating energy cost is required. Since the sludge produced at LGWWTP is still in the Class B categories, primarily due to some metal and pathogen contents, the operating strategies should be revisited to optimize the SRT and VS destruction efficiency. The digester system should provide the operating flexibility to meet different requirements of sludge end products, such as shorter SRT and sufficient sludge blending. In accordance with the future flow and load increases, the centrifuge operating hours need to be extended to handle the sludge production.

2.2.3 Non-Recyclable Residuals

The non-recyclable residuals generated from the Lions Gate WWTP include the screenings and grit. Screenings captured by the 6.0 mm opening mechanical screens and scum screenings (floatable) collected downstream of the bar screens are conveyed to the compactors for dewatering. Due to significant faecal matters and high moisture content, the compacted screenings are hauled to landfill for disposal. Currently, local municipal waste disposal facilities in the GVRD cannot accept this waste stream due to these characteristics. Grit collected from the bottom of grit chambers and pre-aeration tanks is pumped to the grit cyclones and classifiers where the grit is concentrated and washed. The washed grit is hauled to landfill for disposal.

The Contracted Services Division of GVRD Engineering and Construction Department currently administers the hauling and disposal of WWTP residuals. The hauling records (per hauling trip) of the screenings and grit are shown in Figure 2.8 and 2.9, respectively. The digester cleaning work contributed to high screening volume recorded in September 2001. The annual residual wet volumes are summarized in Figure 2.10 (2001~ 2004).

In 2002, total screening and grit productions are about 155 and 220 wet tonnes/year, respectively. The total expenditures of residual hauling were $95,600 plus GST in 2002.

FIGURE 2.8
LGWWTP BAR SCREENINGS PRODUCTION (2001 – 2002)

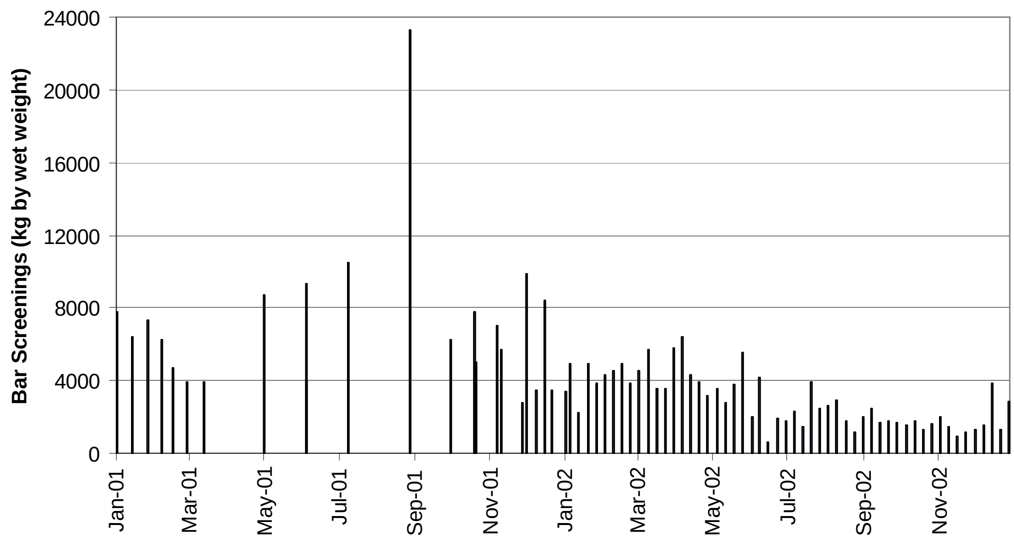


FIGURE 2.9
LGWWTP GRIT PRODUCTION (2001 – 2002)

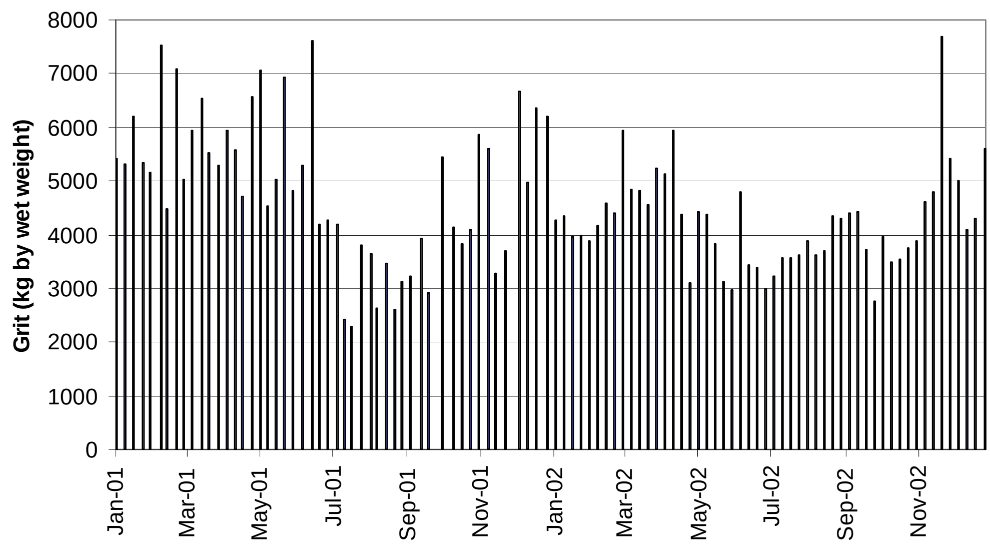
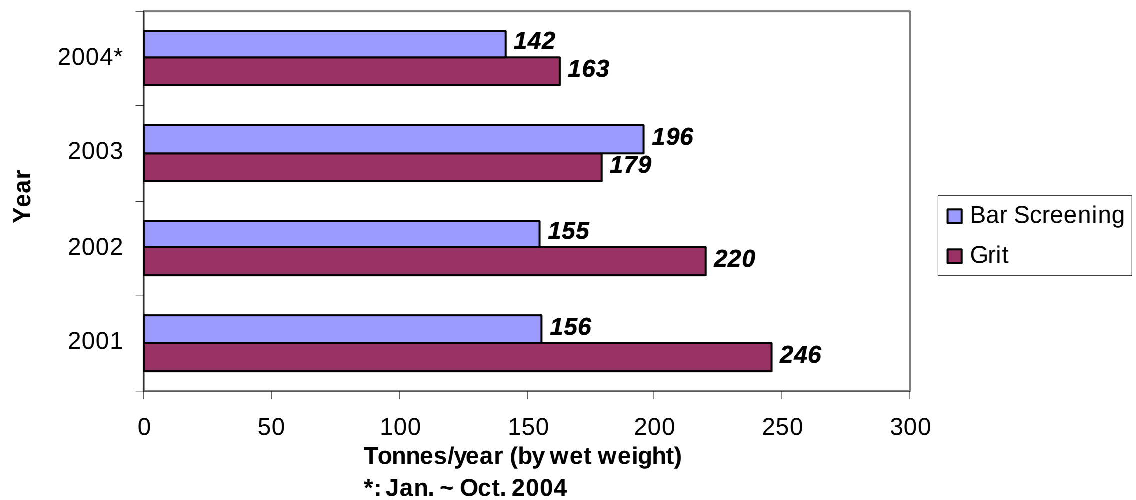


FIGURE 2.10
LGWWTP ANNUAL NON-RECYCLABLE RESIDUALS (2001-2004)



Screenings and grit productions at LGWWTP have been mixed and disposed of at Vancouver Landfill since March 2004. The overall productions of screens and grit have gradually reduced (see Figure 2.10). Such reductions could be caused by the efforts of sewer separation in the catchments and/or reduced removal efficiency due to flow and load increases. Comparing to Annacis Island and Lulu Island WWTPs, LGWWTP removed equivalent amount of screenings as AIWWTP and equivalent amount of grit as LIWWTP on tonnes per ML sewage basis:

Average Non-recyclable Residual Productions (annually wet weight) of 2001 ~ 2004

- LGWWTP: screenings (3.3 tonnes/ML), grit (5.8 tonnes/ML)
- AIWWTP: screenings (3.4 tonnes/ML), grit (3.7 tonnes/ML)
- LIWWTP: screenings (6.8 tonnes/ML), grit (5.8 tonnes/ML)

3 PREDICTED SLUDGE/RESIDUAL QUANTITIES AND CHARACTERISTICS

The sludge quantity and characteristics are one of the critical parameter in the decision to select the secondary treatment upgrade processes. Different treatment processes will produce different types and quantities of sludge, which require different levels of treatment and handling efforts. Most importantly, the sludge quantity and characteristics will affect the capital investment, O/M cost, and recycling options. It is generally preferable to produce smaller volumes of sludge with more manageable characteristics. The following factors are the main considerations:

- Sludge quantity
- Ease of sludge stabilization (i.e. digestion)
- Ease of handling (e.g. dewaterability)
- Fertilizer value for land application or other recycling options

The characteristics of sludge produced by the existing primary plant are described in Section 2.1, as well as the non-recyclable residuals (including scum and grit). The projected sludge quantities for different interim upgrade options, based on the maximum monthly load conditions, are discussed in Section 3.2 and 3.3 for IIWWTP and LGWWTP, respectively.

The interim upgrade options identified in Appendix 3 are listed as follows:

- IIWWTP

  Option 1A: Primary + 50% average dry weather flow (ADWF) conventional activated sludge (CAS)
  Option 1B: Primary + 100% ADWF CAS
  Option 2: Primary + 50% ADWF roughing trickling filter (RTF)
  Option 3: 50% ADWF high rate activated sludge (HRAS) + (Q – 50% ADWF) Primary
  Option 4: Chemically enhanced primary (CEP) + 50% ADWF RTF
  Option 5: CEP only

- LGWWTP

  Option 1: Primary + 50% average dry weather flow (ADWF) biological aerated filter (BAF) in Series
  Option 2A: 50% ADWF RTF + (Q – 50% ADWF) Primary (Parallel)
  Option 2B: 100% ADWF RTF + (Q – 100% ADWF) Primary (Parallel)
  Option 3: CEP + 50% ADWF RTF (Parallel)
  Option 4: 50% ADWF HRAS + (Q – 50% ADWF) Primary (Parallel)

At the current level of treatment, the primary sludge is the only source of raw sludge. With several of the interim treatment upgrade options, biological sludge (wasted activated sludge from suspended growth processes or sloughing biomass from fixed film processes) and chemical sludge (CEP process) will be generated. In the options with CEP, mixed chemical sludge and primary sludge will be produced from the primary sediment tanks. Biological sludge will be produced from the final clarifiers in the options with partial biological treatment (CAS, RTF and TF/SC). Biological sludge generated from BAF process during the backwash cycle can be handled individually or co-thickened with the primary sludge.

3.1 SLUDGE/RESIDUAL CHARACTERISTICS AND QUALITY

Primary sludge, chemically enhanced primary sludge, biological sludge, or their combinations, will be produced from varied interim upgrade options.

Primary Sludge

Primary sludge (PS) is produced in the primary sedimentation process without chemical aids. Generally, PS is gray, slimy, and odorous. Untreated thickened primary sludge typically has a total dry solids concentration about 2 ~ 7 % with 60 ~ 70 % of volatile solids in total dry solids. The amount of dry solids produced ranges from 110-170 kg/1,000m^3 (Metcalf & Eddy, 2003). PS can be readily digested under suitable conditions of operation. Gravity thickening is considered a practical method for primary sludge volume reduction, and 5 ~ 10% thickened PS can be achieved with gravity thickening.

The production rate of PS is subject to the solids removal efficiency in the primary sedimentation process. The improvement of primary sedimentation performance, such as optimization of surface overflow rate (SOR) and add-on sedimentation enhancement (e.g. lamella plates), may also increase the PS quantity. As indicated on Figure 3.1 and 3.3, primary sludge production (dry mass) during the interim stage will increase to about 46 ~ 48 dry tonnes/d at IIWWTP and 12 ~ 15 dry tonnes/d at LGWWTP.

Chemically Enhanced Primary (CEP) Sludge

Chemicals such as ferric salts and alum are commonly used in CEP to enhance removal efficiencies of TSS and BOD. Polymer addition is usually needed to achieve desired CEP performance, however the dosage of polymer needs to be optimized. Sludge produced from CEP treatment is generally darker than PS in colour. CEP sludge is somewhat slimy and may be gelatinous if ferric chloride or alum is used. The odour of chemical sludge is less offensive than the odour produced by PS alone.

The production of CEP is dependent on the chemical dosage and its settling characteristics in chemistry. In comparison with conventional primary sedimentation, significant increase of sludge production is expected with CEP due to the chemical mass and additional removal of primary sludge. CEP sludge usually has better settleability than PS due to the chemical aids as coagulants and higher specific gravity. However, CEP sludge is considered less degradable in the stabilization process due to its chemical constituents. Because they are chemically bound, the nutrients in sludge are less available thus reducing their value for land application and other recycling options.

Biological Sludge

Biomass is grown in biological treatment processes that utilize organic substrates (i.e. BOD in wastewater) as the food source for proliferation. For maintaining an optimum operating condition, certain amount of biomass is wasted from the process to control the biomass growth in bioreactors.

Wasted activated sludge (WAS) generated from the conventional activated sludge (CAS) process is usually brownish, flocculent and inoffensive under good operating condition. If the sludge looks darker in colour than usual, it may be approaching a septic condition and generates offensive odours. If the sludge is lighter in colour, it may not be sufficiently aerated and it settles poorly. WAS can be withdrawn from the bioreactor directly or the underflow of final clarifiers. When the WAS is withdrawn from the final clarifier underflow, typical solids concentration of WAS is about 0.6 ~ 0.8 %. It is a common operation to thicken WAS before digestion either by gravity, mechanical screening or dissolved air flotation (DAF). Thickened WAS concentration can reach 2 ~ 5 % by using DAF subject to the sludge characteristics and polymer usage.

The quantity of WAS is dependent on the solids retention time (SRT) maintained in the bioreactors. Greater amount of WAS is expected with shorter SRT, and vice versa. For example, shorter SRT is operated in HRAS than CAS, therefore, greater sludge production is expected by using HRAS. Biological aerated filter (BAF) process is expected to produce equivalent amounts of WAS as HRAS. WAS in BAF is produced during the backwash cycle, it can be co-thickened with primary sludge in the DAF units, or preferably be handled separately.

The sludge produced in the roughing trickling filter (RTF) process is mainly due to the biomass sloughing from the growth media. In the trickling filter and solid contact (TF/SC) process, biological sludge is produced from both the biomass sloughing (TF process) and suspended growth (SC process). Sludge produced from the RTF and TF/SC is generally brown and flocculent, with inoffensive odour under good operating condition. Trickling filter/solids contact sludge is easily settled and can be readily digested.

The distribution of metal and organic contents in the sludge is the results of physical, chemical and biological reactions throughout the treatment processes, as well as the characteristics of raw sewage entering the plant. It is difficult to predict the metal and organic contents during the design stage. The analytical results prepared in the Biosolids Quality Lab Reports reveal the primary sludge quality at current level of treatment. More metal contents may be expected in the CEP and biological sludge due to chemical binding and biological absorption. The organic contents may be affected by chemical precipitation and biological degradation. However, it is also possible that the metal and organic concentrations in sludge may be diluted by the increases of sludge production in mass (e.g. chemical sludge). Sludge/biosolids qualities (nutrients and metals) are estimated in Table 3.1. Compared to the primary sludge, these nutrients/chemical concentrations are expected to be higher in the CEP and biological sludge.

TABLE 3.1
ESTIMATED SLUDGE/BIOSOLIDS QUALITY

Chemicals/Nutrients (mg/kg dry kg)	Primary Sludge	CEP Sludge*	Secondary Sludge*	Digested Sludge**
Arsenic Total	1~3	N/A	5~10	8
Cadmium Total	1~3	10~20	5~10	3.2
Chromium Total	30~70	300~400	100~150	71
Cobalt Total	2~5	N/A	5~10	4.7
Copper Total	1,000~1,800	3,000~4,000	2,000~3,000	1,360
Lead Total	60~90	400~600	150~200	91
Mercury Total	5~8	N/A	5~8	3
Nickel Total	30~50	100~200	50~100	19
Zinc Total	400~700	1,000~2,000	700~1,500	908
Total Nitrogen	25,000~40,000	28,000~45,000	30,000~50,000	~55,000
Total Phosphorus	10,000~20,000	15,000~25,000	20,000~30,000	N/A

*: in part based on Bonnybrook WWTP, Calgary, 1998
**: Annacis Island WWTP digested combined sludge (2002)

The productions of non-recyclable residuals (screening and grit) will continue in every interim upgrade option. Their quantity will also be affected by any future improvement arrangement, such as dewatering, or upgrade of bar screens and grit chambers. However, no significant increase in grit production is anticipated as a result of the sewer separation program. In fact, as the combined sewers are replaced by separating storm and sanitary sewers, the volume of grit could be reduced.

Screenings

Screenings collected by the bar screens consist of a wide range of coarse materials, which include plastics, rags, paper, hairs, stones and organic solids etc. Their compositions and quality are not homogeneous in nature and difficult to predict. However, it is important to assure the removal of screenings to prevent clogging. Improvement of proper dewatering and storage are also possible to minimize the screening volume. During the preparation of this report, the bar screens at IIWWTP are in the process of being upgraded to improve the operation and efficiency. Fine screens (6 or 9 mm opening) are proposed at IIWWTP for build out to secondary to improve the capture efficiency. If this upgrade goes ahead, substantial increase of screening volume is expected.

Grit

Grit consists of sand, gravel, stones, pebbles and cinders, which have greater specific gravity than water. In general, the grit removals from the gravity settling and aerated grit removal process are predominately inert. The moisture and organic contents of grit are highly varied. Putrid odours are inevitable during processing and storage. The most common disposal methods of grit are landfilling and stockpiling.

Supernatant from Thickener

The thickener supernatant, which carries portions of BOD and TSS loads that cannot be gravity settled in the thickeners, is returned to the headwork wet well. At current high level of thickener removal efficiency, the BOD and TSS loads being returned through the supernatant are estimated less than 1.5% of total plant loads. However, this supernatant load may increase when the thickener capacity is stressed due to the plant flow and load increases.

Hydrolysis and fermentation of primary sludge in thickeners under anaerobic condition may convert organic substances into soluble substance and fatty acids resulting in soluble BOD increase in the supernatant. Currently, SBOD increase due to potential hydrolysis and fermentation in thickeners is marginal, i.e. less than 0.5% of plant total BOD loads. In overall, this supernatant return loads have little impact on the plant removal efficiency and effluent composite concentrations. This organic load can be handled in any future biological treatment upgrade, therefore, side-stream treatment is not recommended.

3.2 IIWWTP PROJECTED SLUDGE/RESIDUAL QUANTITY

The projected sludge production of the interim upgrade options are shown in Figure 3.1 and Figure 3.2, for dry solids mass in Tonnes/d (before digestion) and wet volume (digested and dewatered at 27~ 35% solids concentration) in m3/year, respectively. The projected sludge volumes are based on the average annual loading for BOD and TSS for the upper envelope for loading projections. It should be noted that the sludge volumes indicated in Appendix 3 and 4 are based on maximum monthly loading. Sludge volumes based on maximum monthly loading were needed in order to size the various unit process capacities for solids handling.

As indicated in Figures 3.1 and 3.2, CEP+ 50% RTF produces the most solids in dry tonnes (undigested) and primary + 100% ADWF CAS produced the greatest wet sludge volume after dewatering at 27% solids. It should be noted that at Workshop # 3, the options of primary + 50 RTF (Opt. 2) and CEP + 50% RTF (Opt. 4) have been short-listed. However as described in Appendix 10, the secondary clarifiers in Option 4 could be omitted and the sludge produced and the sludge volumes would then be the same as Option 5 CEP.

By 2021, the screening and grit are estimated to be about 320 Tonnes/year and 1,700 Tonnes/year, respectively, assuming no screening and degritting upgrade during the interim stage. Substantial increases of the screening and grit volumes are expected if the associated unit processes are upgraded (e.g. fine screens and grit removal chambers). However, such speculations may not be valid since the removal efficiencies are subject to the particle size, substance settleability, as well as unit process efficiency. In fact, screenings and grit productions decreased since 2001, which could be the result of sewage separation efforts.

FIGURE 3.1
PROJECTED SLUDGE QUANTITY (UNDIGESTED DRY SOLIDS MASS) OF IIWWTP

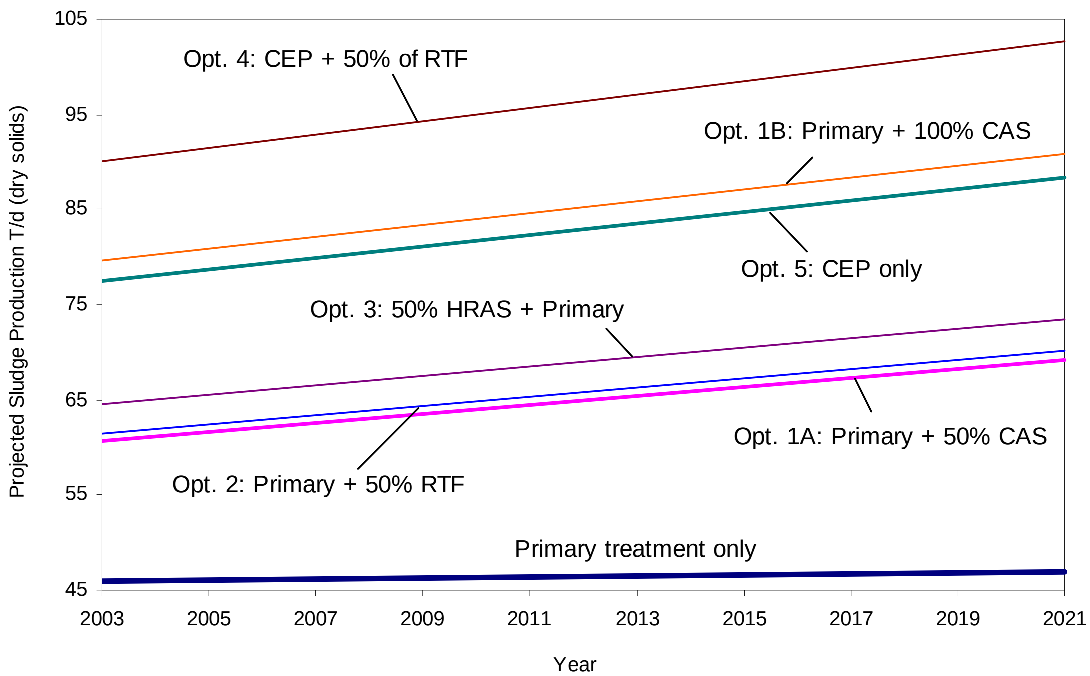
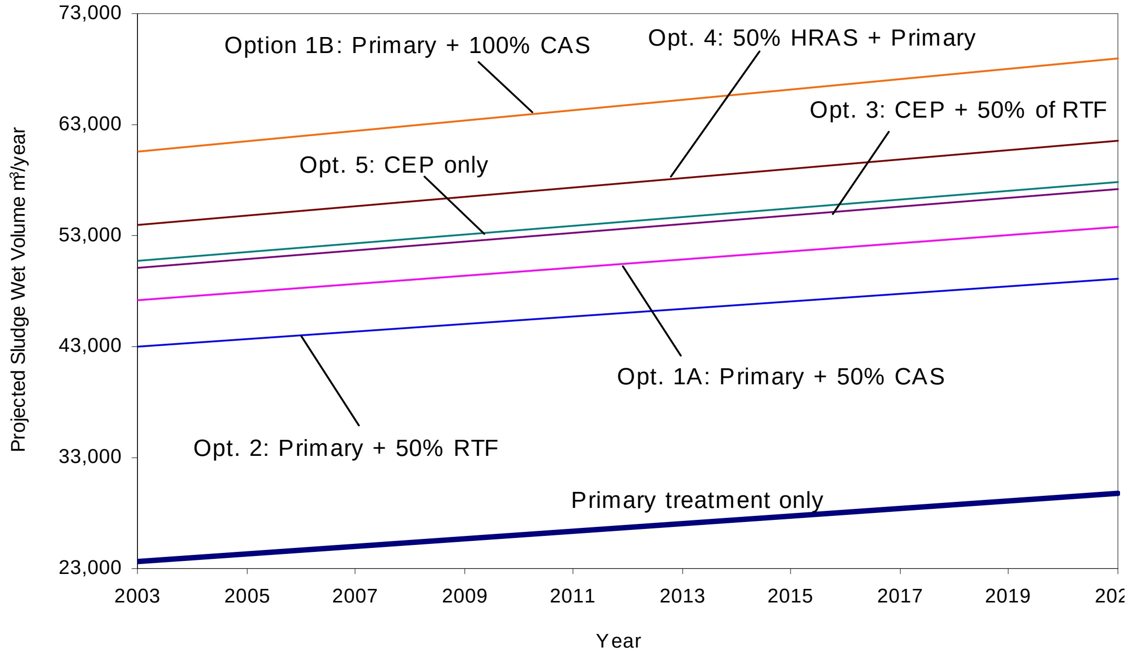


FIGURE 3.1
PROJECTED SLUDGE QUANTITY (WET VOLUME AFTER DEWATERING) OF IIWWTP



3.3 LGWWTP PROJECTED SLUDGE/RESIDUAL QUANTITY

The projected sludge production of the interim upgrade options are shown in Figure 3.3 and Figure 3.4, for their dry solids mass in Tonnes/d (before digestion) and wet volume (digested and dewatered at 27~35% solids concentration) in m^3/year, respectively. CEP+ 50% RTF produces the most solids in dry tonnes and the greatest wet sludge volume after dewatering at 35% solids. At Workshop # 3, the option of primary + 50% ADWF BAF was short-listed.

By 2031, the screening and grit volumes are estimated at about 180 Tonnes/year and 290 Tonnes/year, respectively, assuming no screening and degritting upgrade during the interim stage. No significant increases of the non-recyclable residual are expected due to sewer separation program and possible reduced remove efficiency.

FIGURE 3.2
PROJECTED SLUDGE QUANTITY (UNDIGESTED DRY SOLIDS MASS) OF LGWWTP

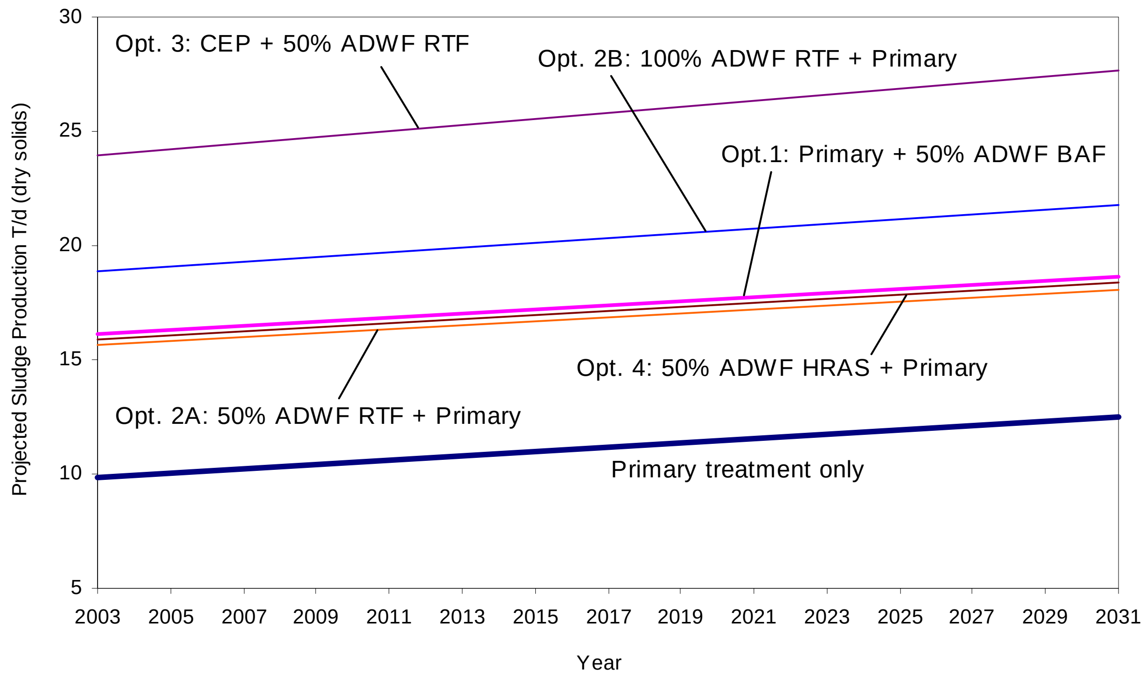
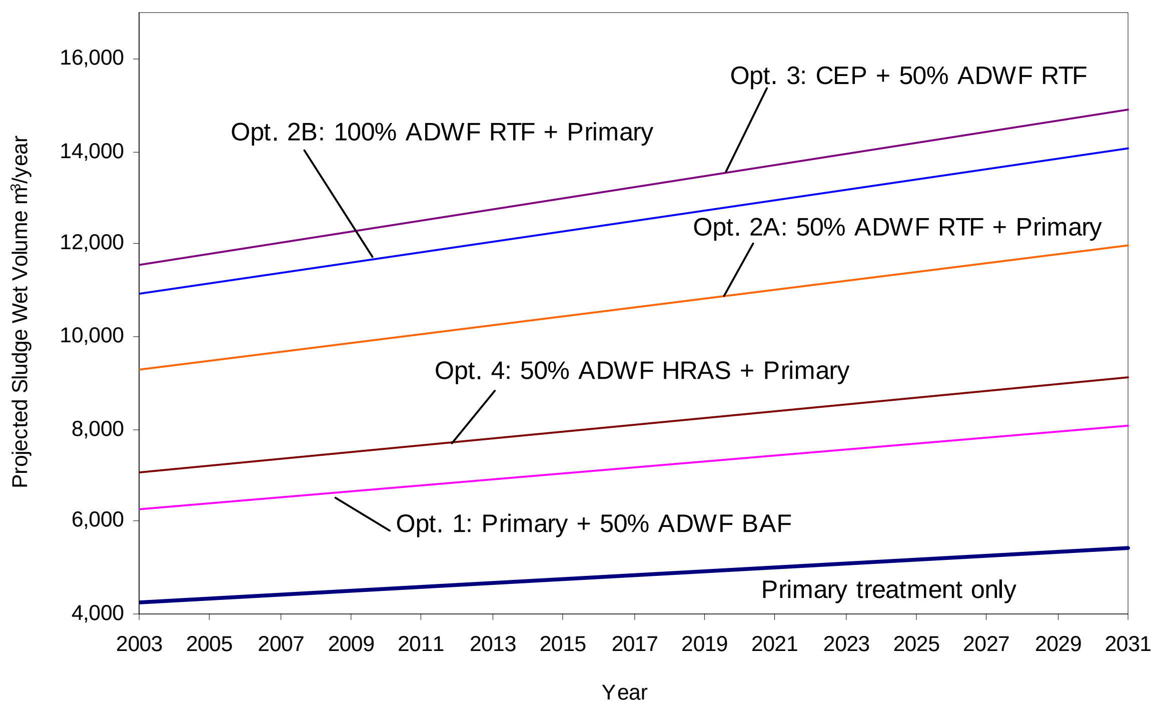


FIGURE 3.3
PROJECTED SLUDGE QUANTITY (WET VOLUME AFTER DEWATERING) OF LGWWTP



4 OVERVIEW OF EXISTING BIOSOLIDS MANAGEMENT PLAN

GVRD is in the process of completing the Biosolids Management Plan (BSMP) by 2007. A biosolids management plan scoping document prepared by the Biosolids management Group (2003 Draft) has highlighted the important issues and challenges in preparing and implementing the BSMP. The steps toward a consolidated BSMP identified in this scoping document including the follows:

- Planning, management and team formation
- Define the goals and analysis frameworks
- Physical handling analysis
- **Treatment and processing technology**
- Source control
- Quality control
- **Market Research and marketing**
- Public communication and consultation
- Education and demonstration projects
- Scientific research
- Contingency planning
- Regulatory requirement
- **Providing environmental, social, and economic values**
- Option decision
- Action plans and performance measurement
- Finalize BSMP

One of the BSMP overall objective is to guide the best decision of the biosolids treatment at WWTP. The recommendations highlighted in the following Sections 5, 6, and 7, including the impacts of the existing sludge handling facility, interim biosolids handling options, and alternative process and emerging technology. Biosolids market is also a key driving force in selecting the treatment technology and options. These issues should be considered in accordance to provide environmental/social/economic sound solutions for biosolids treatment. The sludge upgrades and retrofits of the biosolids processing/handling facilities should provide the flexibility to meets these requirements and demands.

5 IMPACT ON EXISTING SLUDGE MANAGEMENT FACILITIES (INCLUDING SUPERNATANT RECYCLE)

5.1 IONA ISLAND

The sludge quantities in Sections 5.1 and 5.2 are based on the maximum month loading for TSS and BOD. The sizing of the various unit processes for solids handling is based on the maximum month. The annual sludge volume based on average annual loading, please refer to Section 3 of this report.

5.1.1 Thickening

The existing gravity thickeners, with a total surface area of 616 m^2, have sufficient capacity to thicken the primary and chemical sludge for interim upgrades except for the chemical enhanced primary (CEP) options (Options 4 and 5). The projected maximum month primary and chemical sludge productions and volumes (assuming a solid concentration of 5% after thickening) for the interim upgrade options are listed in Table 5.1. Typical design solids loading for gravity thickener is 100 - 50 $kg/m^2/d$ (Metcalf & Eddy, 2003), and 100 $kg/m^2/d$ is selected in this design. The maximum month loads are used to size the gravity thickeners.

The potential impact of CEP operation on the thickener is not clear. It was reported by the plant staff that the thickener return supernatant quality (fine suspended solids and colloids) has deteriorated during CEP runs and resulted in solids concentration increases at primary sediment tank (PST) effluent. The plant has scheduled a full-scale test in 2004 spring to verify this matter and other operating criteria. In addition, the operation of gravity thickeners should be further adjusted and optimized to assure the performance, such as the pumping schedule from PST and underflow withdraw from thickeners.

TABLE 5.1
PROJECTED GRAVITY THICKENER LOAD OF IIWWTP (MAXIMUM MONTH LOADS)

Interim Options	Primary + Chemical Sludge Production (t/d)	Primary + Chemical Sludge Vol. (~5% solids) (m^3/d)	Design Gravity Thickener Load (kg/m^2 d)	Required Surface Area for Gravity Thickeners (m^2)
1A: Primary + 50% CAS	57	1,130	100	565
1B: Primary + 100% CAS	57	1,130	100	565
2: Primary + 50% RTF	57	1,130	100	565
3: 50% HRAS + Primary	28	570	100	283
4: CEP + 50% RTF	106	2,120	100	1,056
5: CEP only	106	2,120	100	1,056

Dissolved air flotation (DAF units) are intended to thicken the biological sludge (wasted activated sludge, WAS), therefore, adequate capacity needs to be provided. Approximately 95% of solids capture rate is expected using DAF for WAS thickening. The subnatant in DAF operation will be returned to the headworks or to the secondary influent for treatment. The projected maximum month biological sludge (waste activated sludge) production and volume (assuming a solid concentration of 3.5% after DAF thickening) for the interim upgrade options are listed in Table 5.2. Typical design loading for DAF thickeners is 30-96 kg/m^2/d for activated sludge without chemical addition (Metcalf & Eddy, 2003), and 48 kg/m^2/d is selected in this design.

TABLE 5.2
PROJECTED DAF THICKENER LOAD OF IIWWTP (MAXIMUM MONTH LOADS)

Interim Options	Biological Sludge Production (t/d)	Biological Sludge Volume (~3.5% solids) (m^3/d)	Design DAF Thickener Load (kg/m^2 d)	Required Surface Area for DAF Thickeners (m^2)
1A: Primary + 50% CAS	27	760	48	552
1B: Primary + 100% CAS	53	1,520	48	1,103
2: Primary + 50% RTF	28	790	48	576
3: 50% HRAS + Primary	60	1,720	48	1,249
4: CEP + 50% RTF	18	510	48	367
5: CEP only	-	-	-	-

5.1.2 Stabilization

The increase in sludge quantity and change in sludge characteristics will have significant impact on the existing digester operation. The projected digester sludge loadings for the interim upgrade options are shown in Table 5.3. A design HRT of 20 days is selected for mesophilic anaerobic digestion. Typical solids loading rate and SRT for mesophilic anaerobic digesters are 1.6-4.8 kg VSS/m^3d and 15-20 days respectively (Metcalf & Eddy, 2003). Existing digesters, with a total volume of 19,820 m^3, will require capacity expansion for interim and build-out to secondary upgrades (See Appendix #3, #4 and for expansion requirements). However, the requirements for expansion and digester operation modes (mesophilic, thermophilic, or others) are subject to the type of end product (e.g. Class A or Class B categorized in BC OMRR for reuse) and if post-treatment (composting, soil amendment etc.) will be carried out following stabilization. Sludge pre-treatment prior to stabilization (see Section 7.1) could rerate digester capacity and defer the digester expansion.

It would be beneficial to upgrade and expand the digester to provide the flexibility of producing different types and/or quality of end products in order to suit the recycling uses and market demands. Further discussions of possible processing technologies and arrangements are discussed in Section 6 and Section 7.

TABLE 5.3
PROJECTED DIGESTER LOAD OF IIWWTP (MAXIMUM MONTH LOADS)

Interim Options	Un-digested Sludge (dry tonne/d)	Digested Sludge Volume (m^3/d)	Digester Design Load (kgVSS/m^3 d)	Required Digester Volume (m^3)
1A: Primary + 50% CAS	83	1,886	2.5	37,729
1B: Primary + 100% CAS	109	2,643	2.5	52,857
2: Primary + 50% RTF	84	1,920	2.5	38,396
3: 50% HRAS + Primary	88	2,278	2.5	45,557
4: CEP + 50% RTF	123	2,614	2.5	52,283
5: CEP only	106	2,111	2.5	42,227

5.1.3 Dewatering

Following the completion of the interim process upgrade (chemical or partial biological treatment), the capacity of the current sludge storage and settling lagoon will reach its maximum capacity within 4 to 8 years for filling at a 0.25 kg VSS/m^2/day. The efficiency of sludge settling (resulting in lower solids concentration) and the quality of return supernatant quality could then deteriorate. Larger stockpiling volume, more frequent sludge dredging, and higher TSS/BOD loads in the lagoon supernatant, and less pathogen kill efficiency are expected when the lagoon loading capacity is exceeded. One of the lagoons may also have to be filled for the use of sludge handling facility expansion, further reducing the total capacity of the lagoons. Mechanical dewatering such as centrifuge is recommended to handle the increased sludge volume. Table 5.4 shows the volume reduction of dewatered sludge and the required number of centrifuges.

TABLE 5.4
PROJECTED DEWATERING LOAD OF IIWWTP (MAXIMUM MONTH LOADS)

Interim Options	Sludge Vol. without Dewatering (m^3/d)	Dewatered Sludge Cake (%)	Dewatered Sludge Cake (m^3/d)	Required No. of Centrifuges (2,000 L/min @35 hr/week)
1A: Primary + 50% CAS	1,886	27%	215	4
1B: Primary + 100% CAS	2,643	27%	279	5
2: Primary + 50% RTF	1,920	30%	196	4
3: 50% HRAS + Primary	2,278	27%	229	7
4: CEP + 50% RTF	2,614	35%	246	4
5: CEP only	2,111	35%	215	4

With a potential secondary treatment upgrade (activated sludge or trickling filters) and anaerobic digestion in place, the dewaterability of biological sludge is considered poorer than the primary sludge alone. The dewatering supernatant characteristics of anaerobic digested biological sludge will result in higher concentrations of TSS, BOD, ammonia, and phosphorus than the primary sludge alone. Since treatment for the removal of phosphorus and ammonia is not required, additional nutrients in the influent is not a concern. However, higher ammonia will impact effluent toxicity limits.

The stockpiling site may not be available in the future due to the space requirement for interim upgrades and build-out to secondary. Finding alternative disposal/land application sites or possibly using the lagoon space for stockpiling need to be planned in details. Potential visual impacts and odour concerns to the adjacent park lands users and residential areas across the North Arm of Fraser River need to be considered as well.

5.2 LIONS GATE

5.2.1 Thickening

The existing gravity thickener, with a total surface area of 147 m^2, performs satisfactory for the solids removal efficiency at LGWWTP. However, there is only one thickener unit and there is no provision for redundancy capability. The thickener capacity would not be sufficient to serve the interim upgrade needs, especially the CEP sludge. Due to the space constraint at LGWWTP, one of the recommendations is to co-thicken primary sludge with biological sludge (e.g. with BAF backwash sludge) with dissolved air flotation (DAF) at higher solids loading rate, and demolish the existing thickener to provide additional space for plant expansion. If the space constraint was not a concern (e.g. the BAF option), it would be possible to add another gravity thickener to provide sufficient capacity and redundancy for the primary sludge. The BAF sludge can be handled separately by DAF with a smaller capacity. The loadings on the thickener for the interim upgrade options are summarized in Table 5.5.

TABLE 5.5
PROJECTED THICKENER LOAD OF LGWWTP (MAXIMUM MONTH LOADS)

Interim Options	Combined Sludge Production (t/d)	Sludge Volume (~3.5% solids) (m^3/d)	Design Thickener Load (kg/m^2 d)	Required Thickener Surface Area (m^2)
1: Primary + 50% BAF	22	620	96	224
2A: 50% RTF + Primary	20	550	96	199
2B: 100% RTF + Primary	22	630	96	230
3: CEP + 50% RTF	28	800	96	292
4: 50% HRAS + Primary	22	620	96	226

5.2.2 Stabilization

Current extended thermophilic anaerobic digestion can achieve a high degree of VS destruction (approximately 70-80% on average) and pathogen kills. However, the existing digester capacity (total capacity of 6,200 m^3 for digesters No. 3 and No. 4) needs to be expanded by an additional 20 to 50% to meet the interim upgrade as shown in Table 5.6. It is recommended to demolish the No. 1 and No. 2 digesters (currently not in use) and to use the space for new digester expansion. The digester system should be capable of been operated to produce different levels of end products, e.g. Class A or Class B for the markets' needs, and to save energy.

TABLE 5.6
PROJECTED DIGESTER LOAD OF LGWWTP (MAXIMUM MONTH LOADS)

Interim Options	Un-digested Sludge (dry tonne/d)	Digested Sludge Volume (m^3/d)	Digester Design Load (kgVSS/m^3 d)	Required Digester Volume (m^3)
1: Primary + 50% BAF	22	615	2.2	9,231
2A: 50% RTF + Primary	19	547	2.2	8,200
2B: 100% RTF + Primary	22	630	2.2	9,445
3: CEP + 50% RTF	28	801	2.2	12,019
4: 50% HRAS + Primary	22	621	2.2	9,317

5.2.3 Dewatering

Mechanical dewatering is considered the best operation due to space constraint at LGWWTP. The reductions of sludge volume by dewatering for the interim upgrade options are listed in Table 5.7. The dewatered sludge at approximately 35% solids can be hauled by truck for land application and recycling offsite. There are currently two 900 L/min centrifuge units operating 35 hours per week and have a total capacity of 800 m^3/d (7 hours per day and 5 days per week). More centrifuge units or longer operating time will be required for the interim expansion to handle more sludge volume. Higher nutrient contents, such as BOD, nitrogen and phosphorus are expected in the return centrifuge centrate due to the digestion of biological sludge. Pre-treatment or flow schedule should be arranged to minimize the impacts of return flow on the treatment system and effluent quality (e.g. ammonia-toxicity).

TABLE 5.7
PROJECTED DEWATERING LOAD OF LGWWTP (MAXIMUM MONTH LOADS)

Interim Options	Sludge Vol. without Dewatering (m^3/d)	Dewatered Sludge Cake (%)	Dewatered Sludge Cake (m^3/d)	Required No. of Centrifuge (900 L/min @ 35 hrs/week)
1: Primary + 50% BAF	615	30%	49	3
2A: 50% RTF + Primary	547	30%	43	3
2B: 100% RTF + Primary	630	30%	50	3
3: CEP + 50% RTF	801	35%	53	3
4: 50% HRAS + Primary	621	27%	55	3

6 OPTIONS FOR INTERIM SLUDGE HANDLING

The sludge handling options for interim operation are discussed in this section, for IIWWTP and LGWWTP, respectively. Their potential impacts on capital investment, sludge quality and quantity, and recycling potential is addressed.

6.1 IONA ISLAND

Lagoon storage/settling and stockpiling are considered the most economical options for sludge handling during the interim stage at IIWWTP. However, because of the limitations of lagoon capacity (based on VSS loading rate), stockpile space requirement and the need to use the area presently occupied by the stockpile for plant expansion, this arrangement will no longer be capable of handling the increases in sludge resulting from the interim process upgrade (e.g. more primary and biological sludge production).

Four (4) sludge handling options have been identified to handle the stabilized sludge (after digestion) during the interim stage:

1. <u>Operate Lagoons and Haul Sludge on a Yearly Basis</u>

   This option will continue to use the lagoons for storage and settling. The settled sludge will be dredged out and dewatered using mobile centrifuges and hauled off-site for stockpiling, or hauled directly to land application sites on a yearly basis without stockpiling. The current stockpiling site will be developed for interim and future process upgrades, therefore, all current land space for stockpiling will no longer be available. Properly scheduled hauling is necessary to ship the biosolids to off-site destinations. With lagoon storage and settling, the wet sludge concentration is estimated about 15% to 20% without dewatering, and at 20 to 30% by mobile centrifuge dewatering. This is less than half of the stockpiled sludge (60~70% solids concentration). As a result of the higher water content of sludge dewatered using a centrifuge, the hauling wet sludge volume will be more than double the operation with on-site drying and stockpiling.

   Currently about one eighth (1/8) of total lagoon volume is transferred to stockpile every year, which results in a nominal retention time of about 8 year in the lagoon system. To ensure the dewatering efficiency and sludge quality, the dredging frequency needs to be increased to accommodate the sludge production increases in the coming years. Therefore, substantial increases of dredging cost and storage/land application space are expected.

   Mobile dewatering devices can be used during lagoon dredging, such as portable centrifuge or belt filter press, to reduce the wet sludge volume and moisture content. This arrangement will increase the operating cost but the sludge volume for hauling can be reduced.

2. Operate Lagoon and Stockpile Sludge Onsite

   This option will continue to use lagoons and on-site stockpiling for dewatering and storage. The dewatering efficiency in the lagoon system will be reduced due to the increase of sludge production in the coming years. Due to the interim upgrade, the lagoon capacity is estimated to be about 3 to 6 remaining years, based on a loading rate of 0.25 kg VSS/m^2· year. The quality of both the dewatered sludge and the return supernatant, will deteriorate when the sludge loading exceed the capacity of the lagoons. Additional stockpiling space will be required on-site and this will result in more wetland being impacted.

   The hauling cost will be the lowest among the other options, since there is no major capital investment and O/M cost associated with this option. The drawbacks will be the degradation of sludge quality and requirement of more stockpiling space. It should be noted that on-site stockpiling is not a complete solution.

3. Abandon Lagoon and Install Centrifuges

   Depending on the interim upgrade option that is selected, the lagoon system have only about 4 to 8 year of remaining capacity (based on VSS loading rates), mechanical sludge dewatering is an option to replace the lagoon and on-site storage for settling and dewatering. Sludge dewatering could be done using centrifuges or belt filter presses. The digested sludge can be dewatered immediately after stabilization in digesters, and hauling can be arranged right after the process with limited on-site storage requirement. The sludge concentration will be about 20 to 30% with centrifuge dewatering. A portion or all of the existing lagoons can be decommissioned and the land can be kept in reserve for future plant upgrades. There will be no need to develop new stockpiling sites. Also, mechanically dewatered sludge generally has a higher nutrient value for recycling uses (e.g. land application) than the lagoon-drying/stockpiling product. However, a significant increased in capital and O/M cost are associated with the installation of mechanical dewatering is immediate.

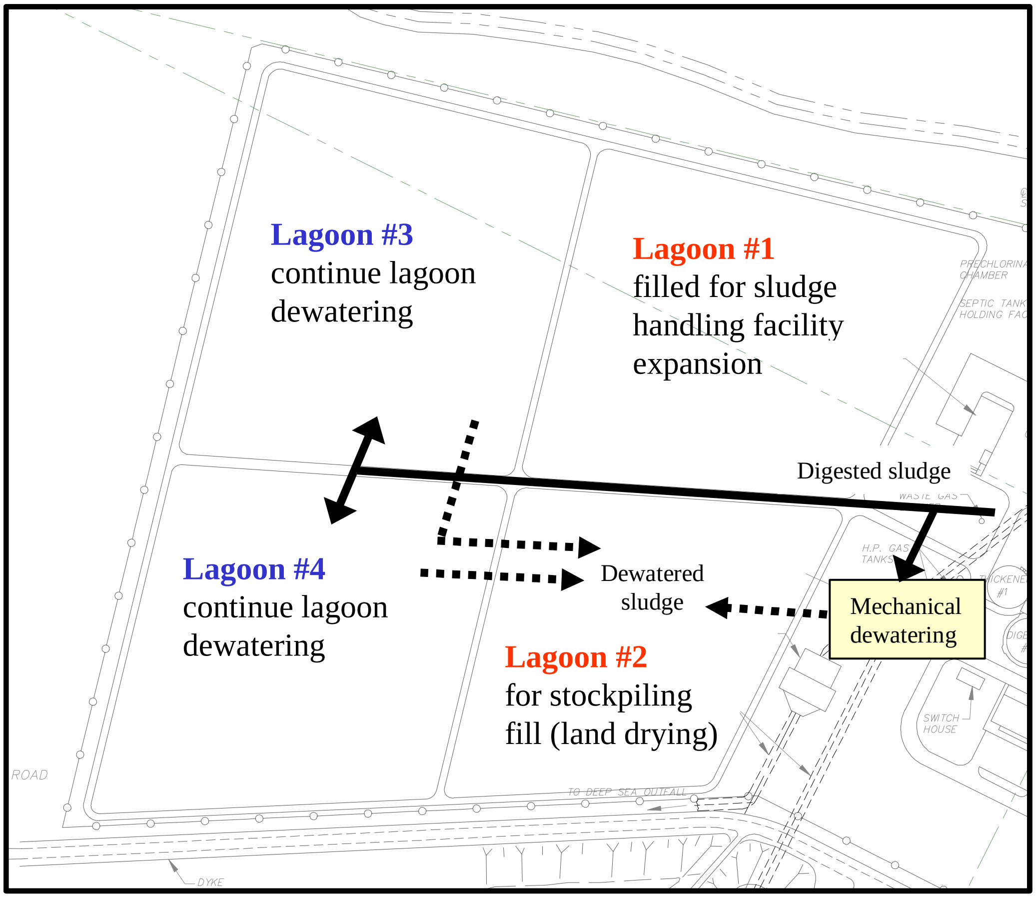
4. Operate Lagoon until 2010 and Install Centrifuge to Dewater Additional Sludge Produced by Interim Upgrade.

   This option will continue to operate the lagoon and stockpiling until the available solids loading capacity is reached before 2010. The actual date when the solids loading capacity is reached will vary depending on the implementation of interim upgrading. Mechanical dewatering will be added to provide additional capacity to handle the excess sludge produced by the Interim upgrades and the lagoons would continue to be used to settle part of the digested sludge. This arrangement can maximize the use of existing system and defer the capital and O/M costs during the interim stage. The additional mechanical dewatering capacity could be expanded as the demand increases in the future. The capital investment can be phased in to provide additional sludge handling capacity or to replace the lagoon/stockpiling entirely in the future. A portion or all of the existing lagoons can be decommissioned and the land can be kept in reserve for future plant upgrades. The disadvantage of this option is that new on-site storage areas must be developed since the plant will be expanded over the areas that are currently used for sludge storage. It is also possible to phase out two of the lagoon operations, and the space can be used for onsite storage.

The selection of the best option for interim and future sludge handling must take into account many factors, including regulatory requirements for sludge quality (e.g. Class A or Class B biosolids), land availability, and disposal/recycling options. Option 3 would discontinue lagoon and stockpiling operations, and allow immediate use of the site for plant expansion. However, the immediate need to find land application sites could be a challenge. Also a substantial increase in capital and O/M costs for the mechanical dewatering are needed to upgrade sludge handling. Option 4 offers the advantage of deferring the immediate need of additional O/M cost and to find new disposal sites. Option 4 provides the maximum flexibility to use the existing facility and produce different quality of sludge end products. Therefore, Option 4 is recommended for interim operation at IIWWTP, and a schematic of this arrangement is illustrated in Figure 6.1

Following the selection of a short list of preferred options for interim upgrades and build-out to secondary, the footprint requirements for site utilization and site preparation (such as pre-loading) have been confirmed. The solids handling Option 4 is refined in Appendix # 10.

FIGURE 6.1
RECOMMENDED INTERIM SLUDGE HANDLING AT IIWWTP



6.2 LIONS GATE

Mechanical dewatering and hauling to offsite locations are considered the most economic option for sludge handling during the interim stage at LGWWTP. Due to the space constraint, there is no extra space onsite for storage or stockpiling. The recommended interim sludge handling strategies are:

- Maximize the process capacity of existing treatment units, including the gravity thickener, anaerobic digester at thermophilic operating condition, and centrifuge dewatering (e.g. extending operation hours).
- Add extra sludge handling capacities, including additional gravity thickener (for redundancy capacity), DAF (for co-thickening), thermophilic anaerobic digesters, and centrifuges.
- Design/retrofit the digester system to be capable of being operated to produce different quality requirements for biosolids recycle options (e.g. composting, pelletization, energy recovery etc.) and land applications (e.g. silviculture and mining site reclamation etc.).
- Investigate the economics and feasibility to operate the digester system in a more efficient mode, e.g. staged operation such as temperature-phases digestion.

7 ALTERNATIVE PROCESS AND EMERGING TECHNOLOGIES

Some alternative processes and emerging technologies can be considered to improve the efficiency in thickening, conditioning and stabilization processes, using physical, chemical and biological methods. Improvements in sludge quality and cost savings can be expected with some pre-treatment and process upgrades. Several post-stabilization processes can broaden the range of potential recycle/reuse options. These alternative processes and new technologies can be classified in four (4) categories, which can be implemented individually or jointly:

1. Sludge Thickening

   By improving sludge thickening efficiency, the solids concentration in the sludge will be increased and raw sludge volume will be reduced. Essentially, the demand of sludge stabilization capacity will be lessened, and the costs of land and capital investment will be saved.

2. Sludge Pre-treatment

   By adding sludge pre-treatment, the sludge characteristics will be acclimatized prior to stabilization. The efficiency of sludge stabilization will be improved by certain pre-treatment. This will result in lower digester volume and the cost of digester expansion will be reduced.

3. Sludge Stabilization

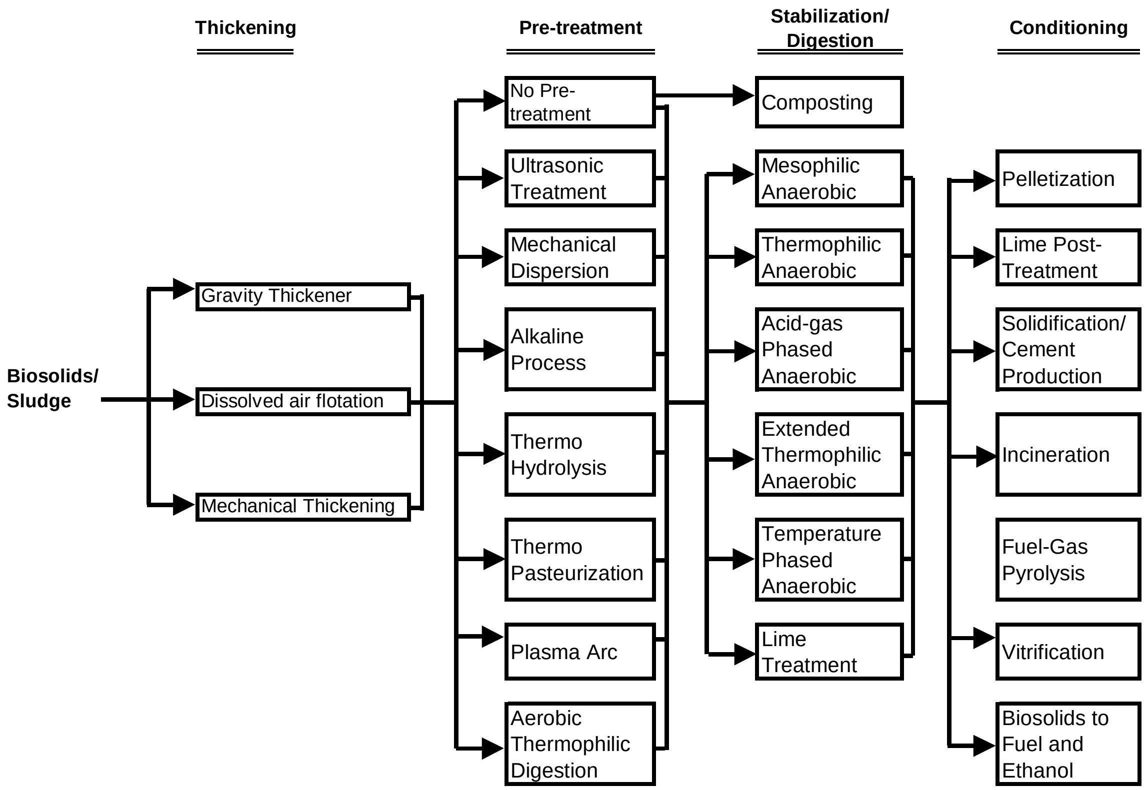
   Sludge stabilization alternatives can be used to improve sludge stabilization efficiency and reduce/defer the needs of digester capacity expansion.

4. Sludge Conditioning for Recycle/Reuse

   GVRD Biosolids Management Group is currently carrying out several studies to explore other recycling/reuse options, which include composting (to produce soil conditioner, growth media etc.), pelletization, lime post-treatment, solidification/cement production, incineration, fuel-gas pyrolysis, vitrification, biosolids to fuel and ethanol. Their processes and potential applications are discussed further in this section.

These sludge process alternatives are illustrated in Figure 7.1 for their potential applications. The final recycling destination will determine the suitable process routes.

FIGURE 7.1
SLUDGE PROCESS ALTERNATIVES



7.1 SLUDGE THICKENING

Primary and CEP Sludge Thickening

Gravity thickener is considered the most economic option for primary/CEP sludge thickening. Within a typical solids loading rate, the thickened sludge concentrations can be about 5 ~10% with primary sludge only. Higher solids concentration can be expected in the case with CEP sludge, in which the chemical (e.g. alum) will act as coagulant aid to enhance sedimentation. Current thickened sludge concentration is about 5.7% at the bottom of gravity thickeners. The thickened sludge concentration can be improved by providing more thickening capacity and reduce the solids loading rate. However, the sludge blanket must be managed properly to prevent septicity in the thickeners due to long retention time.

Other mechanical thickening methods can be considered to further thicken the primary/CEP sludge, such as gravity belt filter and drum thickener. Some emerging technologies, such as Salsnes® Filter, can be considered as an alternative. According to several full-scale operation, mechanical dewatering processes like the Salsnes® Filter

can deliver the thickened sludge up to 20 to 40%. Sludge thickening will have an impact on the selection of the recirculation system for the digesters.
Biological Sludge Thickening

Dissolved air flotation (DAF) is commonly used in large facilities to thicken biological sludge. Centrifuge, screw press, drum thickener are also used but require greater chemical usage. The thickened solids concentration is difficult to predict without a pilot or full-scale testing. The performance of DAF is subject to the sludge characteristics (e.g. SVI) and operating conditions (e.g. air to solids ratio). Typically, DAF can achieve 85% of solids capture for biological sludge thickening without adding polymer. The capture rate can be improved by polymer aids addition. The thickened biological sludge concentration can be as high as 6%, which may result in wet sludge volume reduction by about half (compared to 3.5% thickened solids).

By improving the sludge thickening efficiency, the sludge volume can be reduced significantly. Normally, the hydraulic loading, rather than the solids loading, determines the digester volume. With significant sludge volume reduction, the governing criteria will be switched from hydraulic to solids loading, in which a significant digester volume reduction and cost savings can be expected. For example for interim Option 1A, if the primary sludge can be thickened from 5% to 7% (e.g. mechanical filter) and the biological sludge to be thickened from 3.5 % to 4.5% (e.g. DAF), the sludge digester volume can be reduced by 35%.

7.2 SLUDGE PRE-TREATMENT

Several emerging technologies have been developed at pilot and full-scale levels to adjust the sludge characteristics or reduce the sludge production, particularly by pre-treating biological sludge. The sludge acclimation pre-treatment can speed up the reaction rate in the stabilization process. Reduction of sludge production can also reduce the need for stabilization process capacity expansion.

The anaerobic digestion process involves three distinguishing reactions in sequences: hydrolysis, acidification and gasification. Complex organic molecules in sludge, i.e. the biomass cells, are first hydrolyzed to soluble simple organic substrates. In the following acidification stage, these organic substrates are fermented to simple acids and hydrogen gas by fermentative microorganisms. Methanogenic bacteria will utilize these simple acids and hydrogen gas to produce methane and carbon dioxide in the gasification stage, to accomplish volatile solids (VS) reduction. In these sequential reactions, methane formation followed by hydrolysis is commonly the rate-limiting stage in anaerobic digestion, particularly for the biological sludge. By enhancing the hydrolysis rate as pre-treatment, the overall digestion rate can be improved. This sludge acclimation concept has been adopted in many pre-treatment technologies, which include ultrasonic treatment (Sonix™), mechanical dispersion (Kady Bio-lysis™), alkaline homogenizer process (MicroSludge™), thermo hydrolysis (Cambi™), thermo pasteurization, plasma arc, and aerobic thermophilic digestion (dual digestion) etc.. More descriptions of these technologies are detailed in following sections (6.2.1 to 6.2.7).

Reduction of sludge production can be achieved in secondary treatment process, by means of microbiological and physicochemical methods. Lower sludge yield (observed yield) can be accomplished in membrane bioreactor, extended aeration and high-density oxygen aeration systems, due to their long sludge retention time (SRT) and higher state of biomass hydrolysis. Physicochemical methods to alter the metabolic pathway can also promote sludge reduction, in which metabolic uncoupling agents are added in the bioreactors. Since the extended aeration and high-density oxygen aeration systems are eliminated in the process screening exercise, only the metabolic uncoupling method is elaborated in Section 7.2.9.

7.2.1 Ultrasonic Treatment (Sonix™)

Ultrasonic treatment of sludge is to physically break the biomass cell walls by intensive ultrasonic energy. A frequency of ultrasound around 20 kHz has demonstrated the capability to rupture biomass cellular material and result in more soluble substrate available. This can speed up the reaction of hydrolysis in the following digestion process. The advantages of ultrasonic pretreatment are to improve the sludge digestibility, biogas production, dewaterability (reduce the wet sludge volume), and filamentous control. Sonico (joint venture of Atkins Global and Purac) markets this technology under the trade name of Sonix™ in North America.

The Sonix™ process consists of several in-pipe ultrasound emitters (Figure 7.2) to pre-treat the thickened biological sludge (TWAS). A full-scale operation of 1,440m^3/d sludge volume capacity has been operated for three years in Avonmouth, UK. A full-scale demonstration at the Orange County WWTP, California, has tested the ultrasonic pre-treatment at 5% of TWAS concentration, 17~20 days of SRT and 2.6 kg/m^3/d of VS loading rate in a 4,200 m^3 mesophilic anaerobic digester. Compared with a control digester in parallel without ultrasound pre-treatment, Sonix™ process achieved additional 30% VS reduction of TWAS (from 30~40% to 60~70% of VS reduction). Similar degree of VS reduction improvement was accomplished by treating the mixture of primary sludge and biological sludge. Currently, pilot tests are ongoing at Edmonton, AB and Mangere, New Zealand. A full-scale installation of ultrasonic pre-treatment is scheduled at the Mangere Pollution Control Center (80,000 people equivalent plant) by the end of 2004.

FIGURE 7.2
SONIX™ IN-PIPE ULTRASOUND EMITTERS (SOURCE: ATKINS / SONIC)

Due to the ease of adding the in-pipe ultrasound units, it is considered applicable in both IIWWTP and LGWWTP (no extra footprint required). However, sludge volume reduction and saving of digester capacity expansion need to be justified by economic analysis of capital investment, O/M costs, and disposal expenditure. Pilot test is also recommended to determine design criteria and confirm feasibility.

7.2.2 Mechanical Dispersion (Kady Bio-lysis™)

The mechanical dispersion method applies mechanical and fluid shearing forces including cavitation to rupture cell walls, reduce particle size, disperse particulates, and dissolve materials. This process will reduce the sludge production and make the biological sludge more available in the digestion process. A commercial process developed by KADY International®, under the trade name of BIO-LYSIS SYSTEM® has been demonstrated in several full-scale plants in North America. A process schematic of BIO-LYSIS SYSTEM® is illustrated in Figure 7.3. This technology has also been used for pulp and paper waste reduction.

For municipal wastewater application, this technologies can be used to pre-treat the WAS and TWAS to reduce the sludge volume. Two full-scale pilot tests in Portland, Maine and Detroit, Michigan, 30~50% of sludge volume reduction has been demonstrated. The technology provider also claimed significant improvements of effluent quality and sludge dewaterability. However, there is limited full-scale application experience. Laboratory experimental work with mechanical shearing showed that the digested product was more difficult to dewater.

FIGURE 7.3
MECHANICL DISPERSION (SOURCE: KADY INTERNATIONAL)

7.2.3 <u>Alkaline (MicroSludge™)</u>

MicroSludge™, patented by Paradigm Environmental Technology, is developed to pre-treat the TWAS in alkaline and pressurized conditions. The biomass cell walls are weakening in a pH condition greater than 10 for minimum 60 minutes with sodium/potassium hydroxide additions. Mechanical shear force and sudden pressure relief assist to burst the cell structure, which is called the homogenization process. After sludge conditioning, the pH needs to be neutralized to prevent ammonia toxicity to the anaerobic microorganisms in the digestion reactions. This pre-treatment will improve hydrolysis, which the limiting rate in digestion, the digestion efficiency can be improved and volume reduction can be achieved.

A pilot scale study has been conducted at Lulu Island WWTP using MicroSludge™ for sludge pre-treatment. In the sequential mesophilic anaerobic digestion, the hydraulic retention time was reportedly reduced from 15 days to 5 days, and significant VS destruction and COD/BOD/volatile acids/ammonia reductions were achieved. Dewaterability enhancement and high methane recovery rate were also reported in this study. With MicroSludge™ pre-treatment, it is postulated that the digester capacity can be rerated higher. Recently, Chilliwack WWTP has undertaken the first full-scale prototype application of MicroSludge™ with a design capacity of 4 m^3/hr capacity (co-thickened primary and secondary sludge). The results were not conclusive except to confirm that the sludge characteristics change significantly. Foaming and dewatering problems were identified. A second trial is anticipated at another facility.

7.2.4 Thermo Hydrolysis (Cambi™)

The Cambi™ process is developed by CAMBI AS, Norway, to use thermal energy and pressurization (autoclave) to hydrolyze biomass. A facility picture is shown in Figure 7.4. The thickened sludge (15~ 20% solids) is heated in two pressurized reactors by steam injection at temperatures of 75~110 ºC and 160 ºC, respectively. At these elevated temperatures, the sludge is pressurized to about 30~45 PSI for minimum 30 minutes. The hydrolyzed sludge is then depressurized to force the rupture of cell walls. Process water (e.g. effluent) is added to dilute the sludge concentration to ease the flow through heat exchangers for heat recovery. The sludge is cooled to below 50 ºC and processed in mesophilic anaerobic digestion for stabilization. The process is often referred to as a low rate wet oxidation process (WAO).

Thermophilic hydrolysis produces a large quantity of simple acid, which is easily utilized in the anaerobic reaction. The methogenesis reaction rate is also enhanced to convert acids into methane and carbon dioxide. High degree of pathogen kill can also be achieved in the thermal process to produce Class A biosolids. The gas produced can be recovered to heat the Cambi™ process. Significant digestion rate enhancement and sludge dewaterability improvement have been demonstrated. Hence, the digestion capacity can be increased and digester expansion can be deferred.

Since 1999, many full-scale Cambi™ applications have been operated in Norway, Denmark, Scotland, and England. Their processing capacities range between 8,000 to 36,000 dry solids tonnes/year. There is no full-scale application in North America yet.

FIGURE 7.4
THERMO HYDROLYSIS (SOURCE: CAMBI®)

7.2.5 Thermal Pre-Pasteurization

Thermal pre-pasteurization requires minimum 30 minutes of contact time at a temperature greater than 70ºC. The primary advantage of pre-pasteurization is to achieve high rate of pathogen kills. Mesophilic anaerobic digestion is usually applied following the pre-pasteurization stage. The typical reaction time in the mesophilic anaerobic stage is about for 15 days, which is not much different from conventional single-stage mesophilic anaerobic digestion. Thermal pre-pasteurization followed by mesophilic anaerobic digestion has been successfully operated to meet Class A sludge standards in US and "enhanced treatment" criteria in UK. There are several full-scale applications in North America, including Perris California, Franklin Pennsylvania, and JAMES Abbotsford, BC.

Pre-pasteurization can be easily implemented at IIWWTP to improve the sludge quality. However, no additional digester capacity is gained with this pre-treatment, and significant digester capacity expansion is inevitable. This process is not considered applicable to LGWWTP, since the current digester has been operated at extended thermophilic anaerobic mode.

7.2.6 Thermophilic Aerobic Digestion (Dual Digestion)

Aerobic thermophilic digestion with a short HRT (typically 1~2 days), followed by mesophilic anaerobic digestion, is extensively used in many European countries. This setup is commonly referred as dual-digestion (DD) process. External heat is supplied to elevate the temperature to above 55ºC and aeration is provided to maintain an aerobic condition. Due to lack of available organic substrate in TWAS, endogenous degradation of biomass is the main mechanism to achieve VS reduction. However, the overall HRT (aerobic thermophilic and anaerobic mesophilic stages) is not much shorter than anaerobic mesophilic stage alone. The VS destruction efficiency in dual digestion is not much improved than the anaerobic mesophilic stage alone. The advantage of DD is primarily the pathogen kill rate required to produce Class A sludge. Dewaterability of the DD sludge is also improved.

In North America, CBI Walker Manufacture and Lotepro Environmental System have patented similar processes, using air and high purity oxygen in the aerobic stage, respectively. Full-scale applications in North America can be found in Hagerstown Maryland, Lakawanna New York, and Tacoma Washington. Additional aeration power is required to upgrade the current digester system to DD process at IIWWTP and LGWWTP.

7.2.7 Plasma Arc

Plasma arc pre-treatment applies high voltage pulsed arc-surges to burst materials. Bacterial cell walls are ruptured by plasma energy, resulting in substrate releases from biomass and volume reduction. This pre-treatment can be applied to enhance digestion rate, as well as sludge dewaterability. The most common use of this technology is for hazardous waste treatments, such as medical waste and military blast waste, to achieve high destruction efficiency. However, it is rarely used for municipal wastewater application in full-scale. High-energy consumption and operating safety concerns are the main disadvantages. Demonstration projects in Seattle, WA were also inconclusive. It is not recommended for IIWWTP and LG WWTP upgrade.

7.2.8 Chemical Oxidation: Ozonation and Chlorination

Chemical oxidation by ozonation or chlorination can be used to achieve cell lysis for sludge pretreatment. The lysis treatment can enhance the digestion efficiency and shorten the reaction time required to obtain certain VS destruction. Ondeo® Degremont® has marketed a process to use ozone for TWAS pre-treatment, but the capital/O&M costs (including on-site ozone generation) are considered uneconomic. Chlorination is an economic alternative for ozone, however, potential adverse environmental impacts of chlorination by-products are considered disadvantageous. The chemical oxidation process is not recommended for sludge pre-treatment.

7.2.9 Metabolic Uncoupling

In an activated sludge process, microorganisms oxidize organic substrates in wastewater form new cells and release carbon dioxide and water. During the biosynthesis process, ATP is generated when electrons are transferred from the organic substrate to an electron acceptor such as oxygen (first reaction). The ATP produced is then used in the reverse process to drive photons across a membrane. The later reaction creates a photon gradient across the cell membrane, which is the driving force for the first reaction. This coupling process is known as oxidative phosphylation. If a metabolic uncoupler is added, heat instead of adenosine-5'-triphosphate (ATP) will be produced and the process will be disturbed, thus resulting in lower cell growth. Therefore, metabolic uncouplers such as chlorophenol, 3,3',4',5-tetrachlorosalicylanilide (TCS) and ***para***-nitrophenol (***p***NP) can be used to reduce the amount of sludge production. Studies have shown that the addition of metabolic uncouplers can achieve 50% of sludge reduction or higher.

While this is a promising technology to reduce sludge production, there are still barriers that limit its practical use. First, most metabolic uncouplers are xenobiotic and may be harmful to the environment. Secondly, the use of metabolic uncouplers has shown to increase dissolved oxygen consumption rate by a factor of 50. Currently, there is no such application in full-scale for municipal wastewater treatment.

The following three pre-treatment options are recommended or further evaluation: (1), ultrasonic treatment (e.g. Sonix™), (2) alkaline process (MicroSludge™), and (3) thermo pre-pasteurization. One of these three processes is recommended for sludge pre-treatment at IIWWTP and LGWWTP. Further engineering and economic evaluations are needed to justify the selection of the preferred option.

7.3 SLUDGE STABILIZATION

Temperature phased anaerobic digestion (TPAD), acid-gas phased anaerobic digestion (AGAD), and extended thermophilic anaerobic digestion (ETAD) are capable of achieving high level of stabilization and producing Class A biosolids.

7.3.1 Temperature phased anaerobic digestion (TPAD)

Temperature phased anaerobic digestion is an extension of the single stage mesophilic or thermophilic digestions which incorporates both phases in series. This approach incorporates the advantages of the faster thermophilic digestion rate (generally four times faster) and mitigates the foaming and odours problems commonly associated with single stage digestion. The mesophilic digestion process provides additional polishing and deodorizing of the odour causing compounds commonly associated with the thermophilic process. In addition, TPAD has been demonstrated to be more sustainable to shock-loading than a single stage operation. TPAD can be arranged with a thermophilic followed by a mesophilic process (T/M) or visa-versa (M/T). The mesophilic-thermophilic approach is not commonly applied and limited full and pilot scale data is available. Swiss full-scale results showed the M/T configuration was prone to pathogen regrowth.

A typical thermophilic-mesophilic digestion system utilizes an optimal SRT for the thermophilic process of 5 days (typical range 3-5 d) followed by an optimal SRT of 10 days (typical range 7-15 d) in the mesophilic process. Generally, TPAD systems are operated with a 15 day SRT compared to the 10-20 day SRT for a single stage high rate mesophilic digestion process. VSS destruction in the TPAD system is typically 15 to 25 percent higher than the single stage mesophilic process. Increased gas formation is also possible in TPAD. Operating temperatures are 55 °C and 35 °C for the thermophilic and mesophilic processes, respectively. Maximum VSS loadings have been reported as 4.8 kgVSS/m^3-d, which is about double of conventional single-stage process.

Energy saving can be achieved at LGWWTP, if the operation is modified from current extended thermophilic anaerobic digestion to TPAD mode. However, the efficiency of VS reduction and pathogen kill should be evaluated. At IIWWTP, new thermophilic anaerobic digester for any capacity expansion can be operated in series with the existing mesophilic anaerobic digesters. There would be significant cost savings for IIWWTP and LGWWTP if this type of thermophilic digestion were adopted based upon studies during the pre-design stage of interim treatment.

7.3.2 Acid-gas phased anaerobic digestion (AGAD)

Acid-gas anaerobic digestion occurs in a two-stage process. The first stage is operated at a low pH (6 or less) with a short SRT where solubilization of the particulate matter occurs and volatile acids are formed. The second stage is operated at a neutral pH and a longer SRT to accommodate the methane-generating bacteria. The conditions of the acid phase allows for production of a high volatile acids concentration (typically greater than 6,000 mg/L). The natural pH is an optimum condition for methanogenic microorganism to convert acids to produce methane and carbon dioxide. AGAD systems can be operated at mesophilic or thermophilic conditions. This results in four potential combinations of phased digestion, mesophilic-mesophilic (AGMM), thermophilic-thermophilic (AGTT), mesophilic-thermophilic (AGMT), thermophilic-mesophilic (AGTM).

Typical retention times in acid phase and gas phase are 1~3 days and 10~15 days, respectively. In AGTM process, the VS loading in acid phase and gas phase can be as high as 16~40 kgVS/m^3-d and 6.4 kgVS/m^3-d, respectively. Each process has been applied in various WWTPs throughout North America. The AGMT process has been in operation for the longest (10 years) at the Woodridge-Green Valley WWTP, Illinois. The application of AGTM system in combination with pre-treatment (e.g. SONIX™ and others) can be considered to upgrade the sludge processing system at both IIWWTP and LGWWTP.

7.3.3 Extended Thermophilic Anaerobic Digestion

Extended thermophilic digestion often utilizes a two or three stage process to achieve biosolids reduction. VS reduction has been best achieved through a series of complete-mixed tanks, while plug-flow reactions are best suited for pathogen destruction. To achieve both pathogen and VS reduction both a complete-mixed and plug-flow units are operated in series. Several advantageous of the extended thermophilic process include increased reaction rates, smaller digesters, high VS destruction, higher gas production, higher pathogen kills, and reduced foaming. It has been noted that excellent process control is critical due to the temperature sensitivity of the thermophilic microorganisms and the production of more offensive odours than mesophilic digestion.

Previous performance at the Annacis Island WWTP has achieved a 62% VS reduction at a SRT of 25 days and 55 percent VS reduction at a SRT of 17 days. Currently, the LGWWTP digestion system is operated in extended thermophilic anaerobic mode, however substantial process upgrade is needed at IIWWTP to operate in extended thermophilic anaerobic mode (e.g. digesters, heat recovery, sludge blending/storage).

7.4 SLUDGE CONDITIONING AND DISPOSAL ALTERNATIVES

7.4.1 Lime Stabilization

Lime has been widely used in wastewater treatment plants for alkaline stabilization, either before dewatering of sludge (pre-lime stabilization) or after dewatering of sludge (post-lime stabilization). The purpose of lime addition is to raise the pH level of solids to inhibit microbial growth and minimize odours. The process is capable of producing Class A biosolids, if sufficient lime is added to maintain a pH level at 12 or higher for a contact time of 2 hours. Quicklime (CaO) and hydrated lime ($Ca(OH)_2$) are commonly used. The operation is considered easy and reliable.

With lime stabilization, biosolids produced have reduced pathogens and odours, and are suitable for land application or other beneficial use. However, lime does not remove the organics necessary for microbial growth. Excess lime is usually required to prevent a decrease in pH and regrowth of pathogens. Another disadvantage for lime stabilization is the increased mass of sludge generated, which would result in higher transportation and disposal costs.

The use of lime stabilization is probably not feasible at GVRD because of the lack of market for significantly increased product quantity. This option may be viable if sludge product can be shipped to mining sites for disposal or land acclimation.

7.4.2 Composting

Composting is a process that involves the decomposition of organic materials by microorganisms. The end product is a dark, humus-like material, useful for soil amendments and other beneficial uses. The key factors influencing the process include oxygen, temperature, and moisture.

The process is primarily aerobic, although anaerobic and facultative composting can occur without sufficient air. In the presence of oxygen, the rate of composting is faster and odour is minimized. Composting takes place in two main stages: High rate composting and curing stages. In the first stage, the compost is usually heated to 50-70ºC for thermophilic composting. In the curing stage, the temperature drops, allowing stable and mature compost to be formed. Sufficient moisture content is needed for microbial activities. However, too much moisture will decrease the void spaces of compost, resulting in reduced oxygen transfer. A bulking agent or an amendment such as wood chips and saw dust is often added to the compost to reduce moisture content and increase the porosity of compost for more efficient aeration.

There are three major types of composting systems: Aerated static pile, windrow, and in-vessel composting systems. In the aerated static pile system, dewatered sludge and bulking agent are mixed and placed in large piles, with air blowing into the mixture. The piles are not mixed throughout the composting period. This system has not always achieved OMRR pathogen reduction criteria. The windrow system is similar to the aerated static pile system in mixing and placing sludge and bulking agent in large piles, but differs by providing mixing to the mixture regularly throughout the composting period to enhance aeration. Typical composting period for both the aerated static pile and windrow systems is 21-28 days, and the curing period is approximately 30 days. In the in-vessel composting system, sludge is contained in a reactor, which provides mechanical mixing and aeration. The system can be plug flow, either horizontal or vertical, or dynamic (complete mix).

Implementation at GVRD would require good continuation of available market. Composting may be applied at a small scale, for example 20 tonnes/d, at IIWWTP to diversify products. Anaerobic digestion can be used as a source of biosolids to reduce the capacity of compost operation.

7.4.3 Pelletization

Pelletization is a heat drying process that involves the dewatering of sludge and production of fertilizing pellets. The end product typically has a dry solids content of 90-95%. Heat is applied either directly or indirectly to the sludge in a dryer. In addition to evaporating water from sludge, heat also helps to reduce pathogens and bacteria, thus minimizing the harmful effects of sludge. The advantages of pelletization are to reduce the volume of sludge, reduce the cost of storage and transportation, and increase the marketability of solids. Pellets can be sold in markets and be applied as fertilizers in farms and golf courses. It can also be used in part of topsoil for landfill closure or as fuel for energy generation. Examples of pelletizing plants in US can be found in Boston, Baltimore, Huston, and New York City. Milwaukee has more then sixty years of pelletization operation history.

The sludge pellet products present high risk of fire and explosion when damped, and special attention must be paid during process, storage, and transportation. No stable market demand has been developed in the North America for using the pellet products.

7.4.4 Solidification / Cement Production

Solidification is to use binding materials, most commonly with Portland cement and chemical agents to immobilize the sludge. It has been widely used as a stabilization option for hazardous material handling. The end product can be used as marketable building materials (e.g. in Japan) or safe storage (e.g. in US and Europe). However, solidification will significantly increase the total waste volume, and their long-term risk of waste leaching is uncertain. This is not considered a viable option for GVRD.

7.4.5 Incineration

Incineration is the combustion of organic solids into carbon dioxide, water, and ash. The major advantage of incineration is the reduction of volume of sludge/solids for disposal. The disadvantages are the production of ash and air emission which may be hazardous. Incineration often requires high operating costs. Multiple-hearth and fluidized-bed type incinerators are commonly used for sludge incineration. A trial of multiple-hearth incinerator has been operated at Lulu Island WWTP for years, however this operation has been discontinued due to inconsistent incineration performance. Sludge incineration is getting less public supports mainly due to perceptions of air pollution and ash disposal concerns. Recently, Ontario has planned to phase out sludge incineration and tried to find other alternatives for final disposal.

Incineration can possibly be a good long-term option for GVRD but would require the application of high quality air emission control. It should be combined with waste heat recovery and energy generation (steam or gas). Anaerobic digestion in short-term would still be a viable operation because it will reduce the size of expensive incinerators. However, incineration dose not meet long-term sustainability goals for environmental protection.

7.4.6 Fuel-Gas Pyrolysis

Fuel-gas pyrolysis is a pressurized thermal gasification process to convert hydrocarbons into fuel and gas by partial combustion of waste in absence of oxygen and air supply. The operating temperature is higher than incineration, typically higher than 1,000~1,400 ºC. The fuel and gas produced in the pyrolysis can be recovered as energy sources. Undesired waste gas such as NOx, SO_2 and H_2S are minimized in the gasification process due to high operating temperatures, and dioxin is tentatively to be destroyed. The greenhouse gas productions are also lower than the incineration process. Currently, there are not many large-scale operations for sludge treatment. Complex operation, operating safety, and energy consumption are the main drawbacks.

7.4.7 Vitrification

Vitrification applies high temperature (1,300 ~ 1,500 ºC), commonly by using high electricity between two graphite electrodes, to melt the waste. It is similar to the pyrolysis process with different heating setup. The vitrified waste is sequentially cooled to form non-crystalline, vitreous state solids. Longer cooling times will result in glass-ceramic composites as the end product. Technology suppliers claim volume reduction as high as 99%. Undesired gases and chemicals such as NOx, SO_2, and furans are eliminated in the vitrification process.

The glassified residuals are considered stable matrix, which can be further handle safely. Potentially the glass residuals can be marketed for beneficial reuse (e.g. building blocks). Extensive pre-treatment of sludge is required to remove impurity. The gases produced during the vitrification process provide some energy, which can be recovered for electricity generation.

7.4.8 Biosolids To Fuel And Ethanol

Biosolids to fuel technology is an enhanced thermal conversion to process the sludge into oil or fuel products. Specific pressure, catalysts, and regulated oxygen/air supply, are maintained at higher temperatures to achieve the conversion. Many pre-treatment steps are involved to prepare the sludge as the process feed. The process design and operation are highly dependent on the sludge compositions and characteristics, therefore high level of operating control and optimization are required.

7.4.9 Summary of Biosolids Conditioning and Disposal Alternatives

Lime stabilization, composting and pelletization are considered to be developed technologies and many full-scale facilities are operational across North America and worldwide. Sludge incineration used to be a popular thermal conversion option in many parts of the world, however, newly developed technologies operated at higher temperatures, such as vitrification, have replaced many incineration facilities in several major European municipalities. The operating temperature of vitrification is higher than with conventional incineration, therefore less undesired air pollution and more stable residuals are produced. Biosolids to fuel gas, fuel, and ethanol technologies are generally in developing and innovative stages. There are only few scaled-up applications to date.

The lack of disposal sites and the anticipated low market demand of biosolids-derived products in the future will significantly change the entire business structure for the biosolids reuse program. A long-term biosolids management plan should be able to adjust to changes resulting from market-driven demands and the availability of alternative emerging technologies. The sludge handling facility planning at the wastewater treatment plant should provide adequate flexibility for diversified solutions. A proposed sludge management strategy is discussed in Section 8.0.

8 SLUDGE MANAGEMENT STRATEGIES AND RECOMMENDATIONS

Sludge and biosolids productions are an inevitable end product of wastewater treatment. If beneficial use markets are limited, it is important to implement a management plan to achieve reduction in volumes and provide flexibility for various recycling/reuse options. ,

The following strategies are recommended for interim sludge management at Iona Island and Lions Gate sewage treatment plants.

Sludge Thickening

- Thickening of raw primary sludge to 6 % solids and secondary sludge to 3.5 to 6 % solids will reduce the size and number of the digesters required in new expansions and upgrade of the facilities at IIWWTP and LGWWTP for interim and full secondary treatment. At IIWWTP, the expansion of gravity thickener capacity will be used for primary sludge or CEP sludge thickening. Dissolved Air Flotation (DAF) thickeners are recommended to thicken the biological sludge produced at interim treatment facilities as well as at facilities constructed for build-out to secondary. Due to the space constraint at LGWWTP, DAF units could be considered to co-thicken both the primary and biological sludge. However, co-thickening is not a common practice using DAF. Separate thickening facilities to handle primary/CEP sludge and biological sludge can be implemented if a compact biological treatment process such as Biological Aerated Filters (BAF) has been chosen for upgrade. The small footprint of BAF makes separate sludge thickening possible at LGWWTP, Often for BAF plants, waste biological sludge is co-settled in the primary sedimentation tanks and gravity thickeners can be used to thicken the blended sludge mixture.

- At both IIWWTP and LGWWTP it will be prudent to maximize the use of existing anaerobic digestion and sludge thickening facilities by optimizing the digester and thickening operations, and in the long term implementing sludge pre-treatment. Major capital investment for sludge digester can be further deferred by enhanced sludge thickening efficiency.

Pre-treatment

- Consider implementing sludge pre-treatment to minimize the sludge digester expansion needs, achieve higher solids reduction, and reduce the volume of biosolids produced. The sludge pre-treatment options are only applicable for waste activated sludge (WAS) generated from biological treatment processes.

- The following three options for sludge pretreatment could be examined, probably by demonstrating their application to a portion of the plant sludge at one of the plants:

  i. Ultrasonic treatment (e.g. Sonix™)
  ii. Alkaline process (e.g. MicroSludge®)
  iii. Thermo pre-pasteurization

Sludge Stabilization

- At both IIWWTTTP and LGWWTP, increase the number of anaerobic digesters to expand the digester capacity to keep pace with the increased sludge production resulting from (i) increased population and plant solids loads, and (ii) from provision of interim treatment to control either BOD and TSS compliance excursions or effluent toxicity.

  The sludge management and stabilization facilities should be capable of producing different qualities of biosolids (e.g. Class A, Class B, composting feed, or others). This flexibility would provide the capability to produce suitable biosolids for changing market demands and needs. Any digester capacity expansion during the interim stage could be designed initially for mesophilic anaerobic operation, but should also have the capability of being upgraded to thermophilic mode at minimum cost. The current digesters can be retrofitted to thermophilic mode in the future as the ability to dispose of Class B sludge becomes difficult.

  Currently at IIWWTP, gas produced from the digesters is utilized in the co-generation facilities to produce electrical power, which is subsequently used at the plant for plant needs. These associated gas recovery and electrical generation systems need to be upgraded in the future to accommodate the increase in gas production associated with interim treatment and increased plant loads. Optimized energy recovery should be a design goal for the system expansion designed for the interim treatment and build-out to secondary. At LGWWTP, provision of energy recovery and co-generation will be governed by the feasibility of locating the facilities on the limited site, which will have to accommodate BAF facilities at build-out to secondary.

- It would be beneficial for the GVRD to investigate at a demonstration scale of temperature-phased anaerobic digestion (TPAD) and/or acid-gas thermophilic mesophilic (AGTM) facilities during interim treatment to achieve significant cost savings in the future build-out stage. Most probably this trial could be carried out at IIWWTP because of the multiple digesters at this location and it could be implemented as new facilities are constructed for interim upgrade. As well the current sludge product is Class B (pathogen criteria only) and any sludge product produced during the testing phase would be better than the current Class B and would probably reach Class A through system optimization.

- At LGWWTP, the use of extended thermophilic digestion could be continued as the stabilization process. But new facility construction required by the interim upgrade could be developed as TPAD facilities, which reuse and incorporate the existing digesters in such a manner as to minimize required digester volume expansion and energy saving.

<u>Dewatering</u>

- At IIWWTP, continue to operate the existing lagoons for sludge dewatering and drying for at least a portion of the sludge receiving mesophilic anaerobic digestion. The construction of interim treatment facilities at IIWWTP will occupy a portion of the land currently used for stockpiling of dewatered and dried sludge excavated every 8 years from the lagoons. Additional anaerobic digestion facilities and biosolids dewatering facilities will probably best be located on some of the land currently occupied by the dewatering lagoons (see Appendix 10 proposed site layouts). Since the lagoon area and volume would be reduced by 25 to 50 % to accommodate these facilities, it will be important to phase-in a mechanical sludge dewatering system. The mechanical sludge dewatering equipment would most probably be a centrifuge dewatering system to achieve at least 25 to 30 % total solids. This phase-in could conveniently coincide with the implementation of interim biological treatment and would initially be sized to handle the additional sludge generated by the biological treatment facilities. As thermophilic digestion is implemented (possibly as TPAD or AGTM facilities), the centrifuge dewatering facilities would produce a more marketable sludge end-product.

<u>Alternative Sludge Processing Facilities</u>

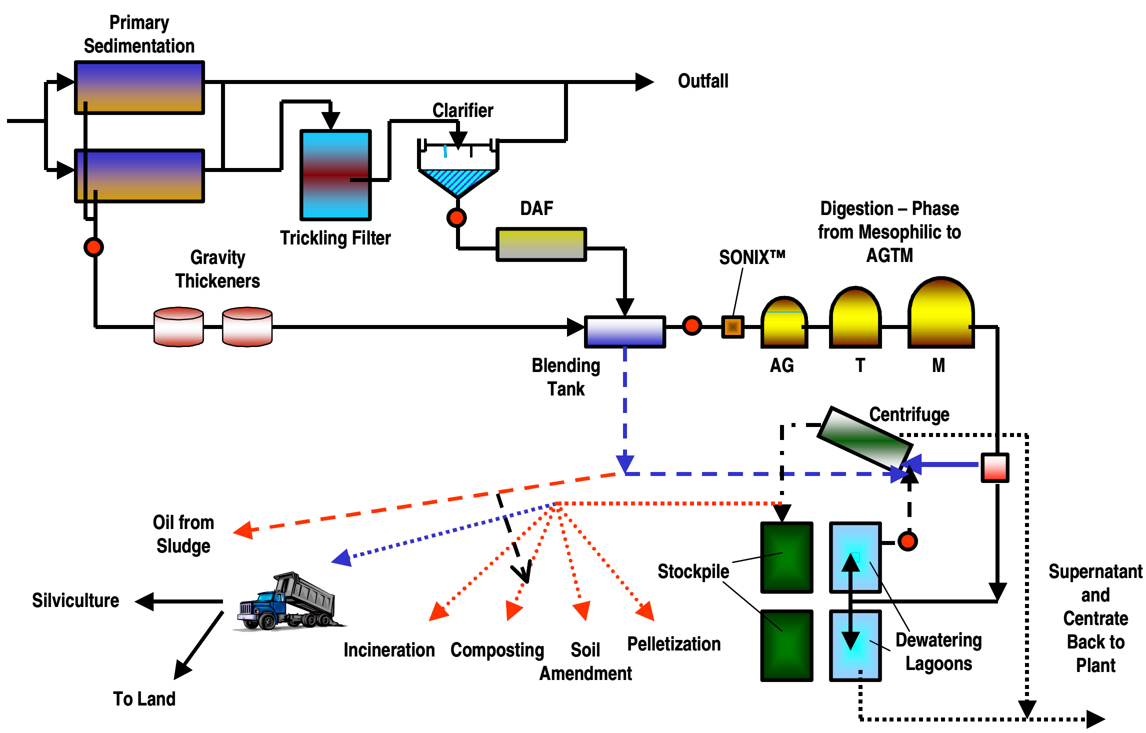
- Land disposal of the anaerobically processed sludge, including Class B from Iona Island, Lions Gate, and Lulu Island, Class A as dewatered (>25% solids) sludge from Annacis Island, is the only ultimate disposal method practiced at GVRD plants for a very significant amount of sludge production. Finding an adequate amount of land for the doubling of sludge quantities that will occur when upgrading becomes necessary. Continuing to enjoy public approval of this method into the future is problematic.

- Many large metropolitan areas of the size of Vancouver are diversifying their sludge products with a portion being applied to land and the rest going to higher reuse options and energy production. A range of sludge management options has been discussed in this Appendix which can be applied in the future.

Potential interim sludge handling processes and arrangements are illustrated in Figure 8.1. The facility planning strategy is to provide flexibility in dealing with future reuse/recycling market demands and processing technologies. Sludge pre-treatment and stabilization should not be precluded because a significant volume reduction can be achieved for cost savings of transportation and handling (e.g. for land application, pelletization, soil amendment, incineration, composting etc.). Dewatering lagoon and mechanical dewatering can both be operated to suit the land application demands and on-site storage capacity. Raw sludge (mixture of primary and biological sludge) can also be processed in compositing and energy recovery applications without digestion.

- It is important that a sludge management master plan be developed which explores the feasibility of alternate and emerging process options to produce a diversified reuse/recycling plan. The master plan should cover the technical reviews, proven applications, cost estimates (capital, O/M, and life-cycle cost), public acceptance, air/water/residual concerns, and product marketability of varied processing alternatives.

FIGURE 8.1
INTERIM SLUDGE HANDLING PROCESS OPTIONS



In summary, for interim sludge treatment/management at IIWWTP and LGWWTP, the following planning approaches are recommended:

Iona Island Wastewater Treatment Plant (IIWWTP)

- Continue to improve and optimize the gravity thickener and mesophilic digester performance.

- Continue to operate the existing anaerobic digesters as mesophilic facilities up to a sludge production of about 63 kg/day, which would coincide with the initiation of interim biological treatment.

- Continue to provide lagoon dewatering, drying, and stockpiling for the mesophilically-digested sludge. Add additional digesters with capability to be operated at both mesophilic and thermophilic modes to accommodate the increased sludge quantities produced by interim biological treatment of a portion of the flow and load.
- The sludge handling capacity and facility expansion of interim upgrade and build-out to secondary options are summarized in Table 8.1 (summary of Appendix 10 for the Design case flow and load condition). Two dewatering lagoons need to be filled during the interim stage, one for solids handling facility expansion and one for on-site solids storage, respectively. At the build-out to secondary stage, a third lagoon needs to be phased out for on-site sludge storage because the existing stockpiling site will be used for facility construction. The approximate year of design, construction, and trial test of TPAD and/or AGTM are also proposed in Table 8.1.

- Add a mechanical dewatering facility (most probably centrifuge dewatering) to handle the interim treatment sludge production as well as the sludge production which cannot be handled by lagoon dewatering, because one or more of the dewatering lagoons is taken out of service to build the additional sludge stabilization and thickening facilities. Demonstration operation of TPAD or/and AGTM can be initiated for a portion of the additional sludge, e.g. on blended sludge equivalent to all of the interim biological sludge production.

- Consider piloting sludge pre-treatment options to defer digester expansion.

TABLE 8.1
SLUDGE HANDLING CAPACITY/FACILITY REQUIREMENTS FOR IIWWTP

YEAR	Interim 2021					Build-out to Secondary 2036	
Parameters	Primary Only - No Upgrade	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Plant Capacity (AAF), ML/d	593	593	593	593	593	593	593
Plant Capacity (PWWF), ML/d	1530	1530	1530	1530	1530	1530	1530
Upgrade Capacity, ML/d	-	114	228	-	228	912	912
Total Sludge production (Annual Average), T/d	48	60	72	91	91	92	105
No. of Gravity Thickener	2	2	2	4	4	2	2
No. of DAF	0	1	2	0	0	3	4
No. of Digester	4	5	6	7	7	8	8
No. of Dewatering Lagoon	4	2	2	2	2	1	1
No. of Centrifuge	0	2	3	3	3	4	4
Approx. Year of Design	-	2006	2006	2006	2006	2016-2017	2016-2017
Approx. Year of Construction	-	2008~2009	2008~2009	2008~2009	2008~2009	2018~2020	2018~2020
Approx. Year of Pilot Trail of TPAD and/or AGTM and Other Alternatives	-	2009-2010				-	

Lions Gate Wastewater Treatment Plant (LGWWTP)

- Improve and optimize the gravity thickener and extended thermophilic digester performance.
- The sludge handling capacity and capacity expansion of interim upgrade and build-out to secondary options, are summarized in Table 8.2 (summary of Appendix 10 for the Design Case flow and load condition). The approximate year of design and construction are also proposed.
- Continue to operate the digester at thermophilic anaerobic condition, mechanical dewatering (centrifuge) and off-site hauling.
- Add additional digesters with capability to be operated at both mesophilic and thermophilic modes.
- Add additional mechanical dewatering facility to treat the increased sludge production generated by interim biological treatment.

- Consider sludge pre-treatment options to defer the digester expansion.

- Convert the extended thermophilic mode of operation to TPAD and/or AGTM operations when proven out by demonstration testing at IIWWTP.

TABLE 8.2
SLUDGE HANDLING CAPACITY/FACILITY REQUIREMENTS FOR LGWWTP

Year	Interim 2031				Build-out to Secvondary 2046
Parameters	Primary Only - No Upgrade	Option 1 CEP ONLY	Option 2A 50% BAF (No CEP)	Option 2B CEP+50% BAF	Option 3 2 x ADWF BAF
Plant Capacity (AAF), ML/d	125	125	125	125	133
Plant Capacity (PWWF), ML/d	300	300	300	300	300
Upgrade Capacity, ML/d	-	-	52	52	222
Total Sludge production (Annual Average), T/d	13	19	24	25	26
No. of Gravity Thickener	2	2	2	2	2
No. of DAF	0	0	1	1	2
No. of Digester	3	4	3	4	4
No. of Centrifuge	2	2	2	2	3
Approx. Year of Design	-	2013	2013	2013	2026-2027
Approx. Year of Construction	-	2014-2015	2014-2015	2014-2015	2028-2030

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GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 8
Current Condition of Treatment Plants

FINAL REPORT

Prepared for

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1 INTRODUCTION

The objective of this report is to evaluate the general condition of the various unit processes, tanks and major equipment in order to determine if a component should be replaced in order to meet the following two conditions:

(1) To integrate the existing primary plant with the proposed secondary plant.

(2) To ensure that the treatment facility can be operated satisfactorily for the next 50 years.

The purpose of this report is not to duplicate the existing operating and maintenance schedule of the plants. The upgrades already proposed in the 10-year Capital Budget Plan are not considered in this report.

The Iona Island plant was originally built in 1962 with upgrades and/or expansion in 1972, 1978, 1983 and 1986. The outfall was upgraded in 1987. The Lions Gate plant was originally built in 1959 with upgrades and/or expansions in 1965, 1975, 1982 and 1990. Some of the issues identified at the start-up meeting include:

- Condition of concrete – age and condition of tanks and how long will they last; corrosion by hydrogen sulfide.
- An overview of seismic compliance and risk assessment. Any change that is proposed to the plants to take into account seismic compliance of existing structures.
- Capacity of unit processes – current hydraulic and solids loading; examine treatment efficiency by reviewing existing performance and removals. This work is covered in Appendix # 3.
- Age, conditions and efficiency of major components/equipment in each unit process.
- Flow distribution of banks of primary sedimentation tanks at Iona Island WWTP. There are 15 primary sedimentation tanks and the GVRD is not satisfied with the current condition of flow distribution.
- Electrical – examine the impact of proposed upgrade on the electrical systems

In addition, the soils conditions have an impact on the performance of a structure during an earthquake. This matter is examined in more detail in Appendix 9 prepared by the geotechnical engineering firm Trow Associates. A brief summary of the soils conditions and their impact on seismic adequacy is included in this report.

2 SEISMIC ADEQUACY

2.1 GENERAL

The following sections provide an overview and seismic assessment of the structures at the Iona and Lions Gate Wastewater Treatment Plants. These assessments were carried out at both plants by Dayton & Knight Ltd. with subsequent overview by Stantec Consulting Ltd.

The GVRD has developed a seismic design criteria which requires that wastewater treatment plants only experience distress, cracking and minor leakage, but otherwise retain complete operational capacity during a 1:475 year earthquake event (Appendix A). This report involved a review of the existing drawings of the structures and applying the current National Building Code of Canada to analyze the structures for a 1 in 475 year return period design basis earthquake.

At both plants, many of the existing structures were designed and built between 1960-1980 and are classified as “Post Disaster Buildings”. Before 1980, code requirements to design water-retaining structures for earthquake conditions were less stringent than the current National Building Code of 1995 and the British Columbia Building Code of 1998. Formulas specified by National Building Codes to design minimum lateral seismic force have two basic factors, which have significant effects on the results. These are the Importance Factor and the Foundation Factor, designated I and F respectively. Before 1980 the factors for post disaster buildings were I = 1.3, and F = 1.3. In recent codes they are now I = 1.5, and F = 1.5 to 2.0. Many of the structures in the plant therefore require to be checked and analyzed for about 30% to 75% more loads than they were originally designed for.

Before the analyses were performed the following assumptions were made:

1. After a 1:475 year design earthquake event:

   a) The tanks must remain usable. Slight structural damage is allowable and insignificant leakage can occur.

   b) The tanks must remain usable, but may suffer repairable structural damages and can be taken out of service, then inspected and repaired in a reasonable time.

	Before 1980	Since 1980
2a.	• design steel strength was 40 ksi (280 MPa)	• design steel strength is 60 ksi (420 MPa)
	• design conc. strength was 3000 psi (21 MPa)	• design conc. strength is 4200 psi (30 MPa)
2b.	Ground acceleration and velocity = 0.20 g.	Ground acceleration and velocity = 0.2 ~0.5 g

3. Capacity/Demand Ratio (C/D)

   The “Capacity” is the structure’s ability to accommodate bending movement and shear. The “Demand” is the applied bending movement and shear forces from seismic forces.

   A capacity/demand ratio less than unity indicates that the structure is inadequate or overloaded. The lower the value of the C/D ratio, the lower is the capacity of the member as compared to the design loads it is subjected to.

Prior to 1985, liquefaction was not typically considered when carrying out geotechnical assessments. The commentary which follows by Trow in the soil condition assessment sections of this report note that liquefaction could occur at these sites with possible vertical movement of up to 250 mm (half of this could be treated as differential over a 5 m distance) and horizontal movement of up to 300 mm. Such movement could cause heavy damage to structures. Some suggested mitigative measures are outlined by Trow. In our structural assessment section, we have noted where liquefaction could cause difficulties. Implementation of liquefaction measures to limit structural movements to 250 mm or less are required to remove possible damage from liquefaction effects.

2.2 IONA ISLAND

2.2.1 Soils Conditions (Summary of Trow Report)

Subsoils at the IIWWTP site consist of deltaic deposits from the Fraser River, comprising unconsolidated silts, sands and silty clays, more than 100 m in thickness, overlying dense to very dense pleistocene glacial soils. The site has been raised using approximately 4.5 m thick river sand fill prior to construction of the existing structures.

The IIWWTP site has been preloaded in several phases prior to construction of the existing facilities. Major portions of the site have been preloaded prior to the original construction over a 2-year period from March 1959 to May 1961. It is understood that preloads with a 2 to 6 month duration were used for the construction of the various additions to the earlier structures. A review of the preload and settlement history indicates that with an 8.5 m high preload, maximum settlement of 1.82 m was observed over a 2-year duration. Post construction settlement as high as 0.7 m was measured over 35 years. Preliminary recommendations for future preloading and setback distance are given.

It is understood that for the seismic upgrading of the existing structures recommendations given in the NBCC 1995 (475 year return period earthquake motion) are to be used. Significantly thick zones of loose sands below the surficial fill zone are expected to liquefy due to the 475-year return period earthquake motion.

Liquefaction would likely cause deformation of the ground, dykes, building foundations and floatation of lightly loaded in-ground tanks.

2.2.2 Structures

The Iona Island Wastewater Treatment Plant is located in one of the highest risk earthquake zones in Canada and all structures in the plant have a high probability of experiencing strong earthquake shaking. Most of the structures in the plant were designed and built before 1980. Before 1980 satisfactory earthquake assessments were not required by code with respect to locations (such as Richmond). The National Building Code of Canada was revised in 1985 and this Code introduced new earthquake design requirements, standards and adequately accounted for seismic forces.

The buildings in the plant, especially the ones built prior to 1980, should be checked, assessed and evaluated carefully according to latest earthquake standards.

An assessment of the Iona Island WWTP structures has been carried out. The plant development has occurred in six stages to date. Assessments of each of the six stages follow.

Stage I

a) Digester #1 and Digester #2 (similar structure)

The digester walls are pinned to the foundation slab and have a semi-contained wall-to-footing connection type. Base shear due to a 1:475 year design earthquake was calculated using the National Building Code 1995 and AWWA Standards.

Friction between the walls and the foundation slab will resist the base shear. C/D ratio is less than unity ≈0.8.

If the walls were to be anchored and contained to the foundation slab the C/D ratio could be more than unity, depending on the anchorage design and detail. The ratio between overturning moment to resisting moment (during the design earthquake) is less than unity. This situation will not overturn the walls, but will move the walls' ring and open up the free joints at the bottom. The digesters will suffer almost non-repairable damage and leak.

Precast panels around the digesters are anchored to the walkway at the top of the digester walls. They are also anchored to a ring slab cantilevered from the mid height of the digester walls. The design earthquake forces could cut the connection and the panels would then collapse.

b) Grit Chambers and Pipe Gallery

These structures have relatively strong reinforced concrete sections. Damage due to the design earthquake and post liquefaction movement is unlikely.

c) Pre-aeration and Sedimentation Tanks

Walls and foundation slabs of the tanks are strong enough to resist the design earthquake. C/D ratios for base shears and overturning moments are more than unity. Vertical and horizontal post liquefaction movements could damage expansion joints, causing leakage, especially at joints in effluent channels between Stage I and Stage V.

d) Sludge Control Building

This is a heavy built reinforced concrete structure. It is strengthened with thick walls and a foundation slab to prevent uplift. Any damage to the building by a 1:475 design earthquake is unlikely.

e) Maintenance Building

This building complies with the present earthquake codes and standards. Therefore damage to this building by a 1:475 design earthquake is unlikely.

Stage II

f) Pre-aeration and Sedimentation Tanks

These tanks are adequately designed and built. The possibility of damage to tanks due to a 1:475 design earthquake is remote. However, post liquefaction movements could damage the expansion joints. Damage to expansion joints in effluent channels, between Stage II, Stage V tanks and between Stage II, Stage IV tanks could be severe. Some stiffening of the roof structure or replacement with a lightweight roof should occur.

g) Sludge Thickener Tank

This tank would be adequate during a design earthquake. But post liquefaction movements could result in the tank suffering uneven movement.

Stage III

h) Digester #3 and Digester #4 (similar structure)

300 mm (12”) thick digester walls have hinged base connections with a 450 mm (18”) thick foundation slab and fixed connections at the top with the dome. The foundation slab thickens to 600 mm (2 feet) thick under the walls and extends 900 mm (3 feet) out from the walls.

Base shears and overturning moments were calculated by using National Building Code 1995 and AWWA standards. C/D ratios for base shear and overturning moment are more than unity. Both tanks will successfully resist a 1:475 design earthquake and will be operational thereafter.

However, the precast outer gallery wall panels around the digester are resting on a ring shaped foundation slab and are anchored to the digester walkways at the top of the walls using clip angles and bolts. Post liquefaction movements could fail the connections and the panels could collapse.

i) Sludge Control Building

It is a strong reinforced concrete structure, partly below the ground. C/D ratio is more than unity. C/D ratio for base shear is also more than unity.

Stage IV

j) Pre-Aeration and Sedimentation Tanks

C/D ratio (moments capacity to demand) is more than unity.

C/D ratio for base shear is also more than unity.

Tanks will be operational after a 1:475 design earthquake.

Stage V

k) Pre-Aeration and Sedimentation Tanks

C/D ratio for walls is more than unity. C/D ratio for base shear is more than unity. After a 1:475 design earthquake all tanks will be operational. However, post liquefaction movement could cause some damage to expansion joints. This may happen especially in effluent channels joint between Stage II and Stage V and between Stage II and Stage IV.

Stage VI

l) Effluent Pump Station

This is a very heavy and strong structure. It will not be damaged during a design earthquake.

The joints between the pumping station and the outfall conduit could be damaged and leak during post liquefaction movement. However they will be repairable.

Conclusions:

- Digesters 1 and 2 will suffer almost non-repairable damage and leaks from the design earthquake.

- Precast panels anchored to Digesters 1, 2, 3 and 4 require some supplementary bracing to prevent their collapse.
- The roofs of Stage II Pre-aeration and Sedimentation Tanks require upgrading.
- Otherwise, the rest of the structures are generally adequate to accommodate seismic forces from the design earthquake.
- Some of the structures in the plant could suffer various types of damage due to uneven ground movements, as a result of liquefaction. Ground surfaces will crack and cracks will run beneath structures. Unless liquefaction mitigation measures are implemented, this could cause damage to structures, especially at the expansion joints between tanks.
- Waterlines, gas lines,, sewer conduits and electrical duct banks in the plant area may suffer some damage and leakage due to differential settlement. Particularly the joints of pipes and outfall conduits, resting on a dyke, may suffer damage due to liquefaction movements. Also of concern is the 12 kV service from BC Hydro and the interconnection between the EPS and the main plant.

2.3 LIONS GATE

2.3.1 Soils Conditions (Summary of Trow Report)

Subsoils at the LGWWTP site consist of: a maximum 1.8 m of FILL comprising sand and gravel with pieces of wood, debris, and organics; a 13 to 15 m thick layer of sand and gravel with some cobbles and boulders; a 25 to 35 m thick layer of silty sand with some gravel; a 20 to 40 m thick very dense glacial till overlying claystone bedrock.

For the design 1:475 year return period earthquake motion, potential liquefiable zones at the LGWWTP site are expected to be scattered sporadically throughout the site, with some local zones of significant liquefaction. Earthquake shaking together with subsoil liquefaction would likely to cause ground settlement and movement towards Burrard Inlet.

2.3.2 Structures

The plant development has occurred in three stages to date. An assessment of these stages follows.

Stage I and Stage II

a) Influent Pump Building (Stage I Drawing S205)

This is a reinforced concrete below ground structure. To prevent the uplift of building, relatively thick and heavy reinforced sections were used. This enables the building to resist earthquake forces successfully. The Capacity/Demand (C/D) ratio for walls and footings is more than unity.

A 1:475 year design earthquake may damage the roof's pre-cast purlin's welded connections and steel framed glass components. However, they are repairable and the pumping station will remain operational.

b) Pre-Aeration and Sedimentation Tanks

i) Base and Walls

Stage I tanks No. 3 and 4 have overall dimensions of 51.8 m (170') x 12.2 m (40') without any expansion joint. Stage II tanks No. 5, 6, 7 and 8 have overall dimensions of 51.8 m (170') x 30.5 m (100') without any expansion joint.

Structural design codes require continuous expansion joints through structures to minimize uncontrolled cracking and to control the location of movements. In walls and structural slabs, expansion joints should be located not more than every 20-25 m (60' – 80'). Walls of the tanks are fixed to the foundation floor and to the walkways at the top. This provides frame action in every direction. Wide continuous walkways act like a diaphragm to distribute lateral forces to walls and footings. With the relatively wide walkways, and 300 mm (12") and 250 mm (10") thick wall sections, these have significant inertia. Average 4.5 m (14') high walls can resist earthquake movements successfully. The C/D ratio for walls and base shear is more than unity.

ii) Roof of the Tanks

The size of the precast columns supporting the roof at the top is 10" x 12" = 120 sq.in. and at the bottom is 10" x 10" = 100 sq.in. The design codes require min. column size to be 10" x 12" or 11" x 11" min. 120 sq.in.

Existing reinforcing in the columns is 4 - #5 = 1.24 sq.in. According to our analyses columns require min. 4-#8 = 3.16 sq.in. steel area. C/D ratio for all roof support columns is less than unity.

Furthermore all welded joints between precast columns, precast beams, purlins and precast box sections should be carefully inspected and repaired if necessary. All precast box sections should be removed and replaced with light roofing materials like metal cladding.

c) Operations Building

Reinforced concrete framed – one level structure. This will resist earthquake movements successfully, but some light damage will likely occur in wide glass windows and support of control room panels. They are repairable and the building will be in service after the design earthquake.

d) Digester #1 and #2

After analyzing the original drawings of the digesters according to force level as specified in the N.B.C. 1995 with I = 1.5, F = 1.3, R = 1.5 and surface PGA (peak ground acceleration) of 0.25 g, we conclude that the C/D ratio for the base shear for vertical seismic forces is less than unity and the C/D ratio for overturning moment is also less than unity. These structures will not survive the design earthquake.

Stage III

e) Pre-Aeration and Primary Sedimentation Tanks

i) Base and Walls

Pre-aeration and sedimentation tanks, walls, walkways and foundation slab with required reinforcement will provide adequate frame action to resist movements in every direction. C/D ratio is more than unity.

ii) Roof Structure

Roof slabs and supporting columns with existing reinforcement, will provide adequate frame action to resist lateral loads and shear at the bottom of columns and at the joints between columns and slabs.

After analyzing the roof slabs and the columns for lateral seismic forces the C/D ratio was found to be more than unity. However, after analyzing the roof for vertical seismic forces a C/D ratio of 0.8 or less than unity for slab sections was found.

Bayline (F, E x 4.5 – 1.2) on Dwg. S361 and Bayline (P, N x 4.5 – 1.2) on Dwg. S362.

We recommend that the above-mentioned slab sections be supported with additional steel beams and pipe columns

f) Chlorine Contact Tanks

Cantilevered walls from the foundation slab and foundation have sufficient capacity to resist earthquake movements. The C/D ratio for walls and base shear for tanks is more than unity.

West and east sides of tanks have uneven backfill. The west side's backfill is almost 3 m (10') higher than the east side. This situation creates an unbalanced pressure and uneven stress in the structure's walls and foundation slabs. To prevent damage to the tanks from this situation during a seismic event, the backfill loading should be equalized.

Existing cross walkways are not anchored to the sidewalls. They are only anchored to the center wall. We recommend that all cross-walkways be anchored to the tops of the sidewalls.

g) Digester #3

We understand that there was some work done to this digester to thicken the original 200 mm thick foundation slab and to reinforce the bottom of the sliding walls. Further details are required before an assessment can be made on this structure's adequacy.

h) Digester #4

Roof dome, walls and foundation slabs have a C/D ratio close to unity. The suitability of this structure depends on the factors used in the analyses recommended by N.B.C. 1995.

Stage IV

i) Sludge Dewatering Facility

This heavy reinforced solid concrete structure was designed and built in late 1990 and 1991. The C/D ratio for all parts of the building is more than unity.

Conclusions:

- Digesters 1 and 2 will not survive the design earthquake.
- Digester 3 should be checked in more detail to confirm that some reported upgrade improvements have occurred.
- Digester 4 is marginal.
- There are 3 tonnes and 3.5 tonnes precast panels around Digesters #3 and #4 resting on a ring shaped foundation slab and anchored to digester walkways at the top using steel angles and bolt connectors. Vertical and horizontal movements due to post liquefaction could fail the rigid connections at the top of the panels and welded connections between the panels and may cause them to collapse.

- The pre-aeration and primary sedimentation tank roofs should be upgraded.

- Unless liquefaction mitigation measures are implemented, most of the expansion joints could become damaged. Vertical movements can rupture PVC waterstops in joints or destroy the bond between the waterstop and the concrete, especially the expansion joint in the effluent channels between Stage I and Stage III tanks. There could be significant damage unless some mitigating action is taken.

- Post liquefaction movements could fracture the connections in all pipelines. Especially the 750 mm ø and 600 mm ø pipelines between the pumping station and pre-aeration tanks.

3 CONDITION OF EXISTING PLANT

3.1 CONCRETE TANKS AND STRUCTURES

3.1.1 Iona Island

Stantec carried out a walk-through and visual inspection of the Treatment Plant structures on September 24 and November 5, 2003. The original design drawings were available for review through GVRD records. No tests were performed on equipment or materials. This report offers comments arising from observations and discussion with the maintenance staff during this review.

- Most of the Treatment Plant buildings, tanks and pipe gallery appear to be in good condition without any major cracking in concrete, buckling of the steel structure or other sign of distress. This implies that the structure as a whole is sound and stable with no significant differential settlement. A summary of the condition survey is listed in Table 3.1.

- Damage occurred recently to the concrete structure at the northwest corner of sedimentation tank No. 5. A large piece of concrete spalled off the exterior wall, exposing the reinforcing steel. At the adjacent catwalks, the steel railing kick plate buckled and a few railing posts were found broken off from their concrete base. A number of steel checker plate covers over the pipe trench were found bent and jammed within the support frame. In the pipe gallery, below this area, only minor hairline cracking and thin concrete surface spalling were noted. No sign of leakage was found at these cracks. The concrete cracking and steel buckling were probably the result of excessive differential settlement between tank No.5 and No. 6. Immediate repair to the concrete is not required. However, the buckled steel checker floor plate should be removed or trimmed to relieve the built up stresses. Further monitoring and investigation is recommended.

- Significant deterioration of the underside of the slabs enclosing the effluent channels was noted. The cement binding the fine aggregates appears to have disintegrated exposing the coarse aggregate. This is thought to have been caused by hydrogen sulphide attack. Previous remedial attempts by installing a protective coating appear to have been unsuccessful. There were no signs of exposed reinforcing steel or significant cracks in the concrete slab or wall. Further monitoring is recommended.

- The expansion/control joint sealants have performed quite well considering their years of service. Some of these were damage due to the tank movements or frost jacking of the perimeter slabs. They can easily be replaced in sections and when required, during regular maintenance.

- One large crack in the exterior concrete slab at the effluent pumping station was noted. The crack may be the result of excessive ground settlement. Further investigation is required.

TABLE 3.1
IIWTP SUMMARY OF CONDITION SURVEY

	Structure	Age(Yr Built)	Material	Code Compliance	Condition	Current Repair
1	Adm. building	1960	Conc./Block	NBC 95	Good	Seismic Upgrade 1998
2	Power house	1960	Conc.	NBC 95	Good	Seismic Upgrade 1998
3	Annex	1998	Conc.	NBC 95	Good	
4	Pipe Gallery	1960-1986	Conc.		Good, minor cracks & seepage	
5	Digestor No. 1	1960	Precast Conc.	*B	Good	Further investigation req'd.
6	Digestor No. 2	1960	Precast Conc.	*B	Good	Further investigation req'd.
7	Sludge Control Bldg. No. 1	1960	Conc.		Good	
8	Sludge Thickener No. 1	1986	Conc.		Good	
9	P & S Tanks No. 1 -5	1960	Conc./Block		1. Conc. crack, spalling (see photos) 2. Effluent channel concrete scaling *A	Further investigation req'd.
10	Pump Bldg. & Grit Chambers	1960	Conc.	NBC 95	Good	Seismic Upgrade 1997
11	Screen Hooper Bldg	1995	Metal		Good, some rusting	
12	P & S Tanks No. 6 - 10	1972	Conc./Block		Good	
13	P & S Tanks No. 11 - 13	1983	Conc./Block		Good	
14	P & S Tanks No. 14 - 15	1986	Conc./Block		Good	
15	Digestor No. 3	1978	Precast Conc.	NBC 95	Good	Roof Replacement 1999
16	Digestor No. 4	1978	Precast Conc.		Good	Sidewalk replaced 2002
17	Sludge Thickener No. 2	1978	Conc.		Good	
18	Effluent Pump Station	1986	Conc.		Good, large crack in slab	Further investigation req'd.
19	Maintenance Bldg	1960	Conc.		Good	

Note *A : 1) Concrete crack and spalling probably due to differential settlement between tanks.
2) Heavy scaling of conrete on wall and channel ceiling probably due to attack by the trapped H2S gas.
*B : structure does not meet the latest ACI350 and NBC code seismic requirements.

3.1.2 Lions Gate

There are minimal visual signs of settlement or corroded concrete in the tanks and structures. However the following comments from the operators and from the visual inspection should be noted:

- The thickener has settled differentially by approximately 150 mm. It is not clear if this settlement has now stabilized. In conjunction with this settlement the main pipe feeding the thickener caused the wall of the thickener pump room to crack. This pipe has since been replaced using additional flexible couplings and the wall repaired.

- The waterstops in the chlorine contact tank leaks, as is evident when one side of the tank is drained and the other is full. This leakage is not severe.

- There is some surface corrosion in the grit auger channels and some H_2S surface attack of the concrete along the waterline of the grit and primary sedimentation tanks. The condition of the influent distribution channel was not inspected but this channel is assumed to be in good condition with some similar surface damage.

- The floor of digester No.3 lifted during the present retrofit work. This has been repaired.
- The concrete roof and floor for digester No.3 is new and the concrete walls are in good condition.
- The concrete on the inside of digester No.4 is unknown
- The grit cyclone enclosure and piping is corroded and should be replaced with a new structure.

3.2 SUITABILITY OF PLANT EQUIPMENT FOR INTERIM UPGRADES AND BUILD-OUT

3.2.1 Iona Island

A site visit was carried out on March 18 and 19, 2004 with the operating staff to examine the current condition of the treatment equipment and unit processes. The purpose of the evaluation was not to prepare a maintenance and replacement program but to determine how the current primary treatment plant could be integrated with the proposed secondary treatment processes. The upgrades and improvements discussed in this section should be considered within the context of the continued use of the primary plant for another 50 years and to ensure that the primary plant can be successfully integrated with the proposed secondary treatment which will consists of trickling filters or biological aerated filters. The observation made during the site visits are detailed hereafter.

Influent Screens and Siphon Discharge

- Influent screens – high velocity of siphon discharge and potential for damage to screens (12 mm bar spacing).
- GVRD is on the third generation of screens. Previous two commercially available screens have now been replaced with GVRD’s own design, heavy-duty bars and “tilt up” out of channel arrangement to help maintenance.
- High velocities from siphon discharge especially at higher flows force grit into the crevices on screen and also force screenable materials through bars.
- Carryover of rags and screenable materials creates many maintenance/operating problems, including buildup in sludge handling systems since the influent screens allow a lot of these materials to pass through.

- Influent flows occasionally exceed pump station capacity. Wet well levels reach elevation where influent gates automatically close to isolate pumps/screens from siphon discharge. The excess flow bypasses plant to outfall pump.

Influent Pumps

- The pumps are 40 years old; upgraded with new impellers, motors and variable frequency drives (VFDs); performance testing by GVRD indicates pump output is around:

  - Large pumps - 140 cfs (342 MLD)
  - Small pumps – 90 cfs (220 MLD)

- Impellers will need periodic replacement due to grit wear, and maintain performance;

- Plant personnel note the pump casings are still the original casing; pump replacement may become an issue to identify in the present study. Any pump replacement must include accessory piping and valving, etc.

Longitudinal Grit Tanks and Pre-aeration Grit Tanks

- These tanks are 40 years old, and are part of original Stage I construction.

- The overall grit system is a two-stage system which includes the longitudinal grit tanks followed by the aerated grit tanks. In between the two, grit settles in the influent channel at lower flows.

- The upgrading of the grit removal and handling system is described in the August 2002 Dayton & Knight study. This report is being reviewed by GVRD and the recommendations may be amended.

- GVRD has budgeted $2.0 million for 2007 grit system upgrade (part of 10 year Capital Budget Plan). Other upgrades to the grit improvements systems will be determined after the August 2002 report by Dayton & Knight has been reviewed.

- A cursory review of the large grit tanks confirmed the estimated peak hydraulic capacity (applied peak design flow of 0.37 m/sec), based on field checks of apparent high water levels in tanks;

- Field observation of one out-of-service grit tank found concrete wall surface showing exposed aggregate and loss of fines cement from visible surface areas. As would be expected, this is seen over the whole tank surface but is more pronounced over the range of water levels fluctuation.

- Depending on flow rate, water surface elevation can rise around 1.2 to 1.5 metre above the minimum flow levels (fluctuates 1.2 to 1.5 metre). This is typical in all tanks, channels observed during the field visit.

Influent Channel and Flow Distribution

- Discharge velocities at the outlet of the longitudinal grit tanks and into the influent channel are quite high due to small area of available orifices. When the plant experiences higher flows, the proportional weir comes into play (estimate over 1 metre/sec), which creates turbulence and elevated water levels in this area. The influent channel is about 4.3 m wide at end of grit tanks, but reduces down to 2.1 m wide on either side;

- The Influent openings in the concrete wall from the influent channel to pre-aeration/primary sedimentation tanks have no control. Each influent opening is fitted with a “window” gate (aluminum plate with rectangular opening). The dimensions of window openings vary widely as documented in the Hay & Company report (July 2002). The window openings are documented in Appendix 4 Weir Gate Survey of the report.

- According to operation staff, there is no documented procedure for use of these window gates, and placement of the different sizes is inconsistent with hydraulic conditions in the channel and being able to split flows reasonably evenly across the in-service sedimentation tanks.

- Further investigation and hydraulic analysis would be required to produce a protocol for sizing and placing specific gate openings to improve flow splitting across the tanks.

- Solids and grit deposition is found mostly in the north and south ends of the influent channel where the velocity is lower. The upgrading of complete (full length) channel aeration system is part of the Dayton & Knight report recommendations (1998). The scope of work could be modified to address the most affected area.

Primary Sedimentation Tanks (PST)

- Scum collector actuators are a maintenance issue. There are old pneumatic type actuators and some actuators that need to be repaired or replaced. The malfunction of scum collector actuators can lead to temporary buildup of scum on tank surfaces. In turn, this buildup can lead to carry over of scum under the scum troughs and into the effluent channel.

- Screenings floatable as well as scum end up passing through the sedimentation tanks, and can be seen in the first rows of effluent launders. This situation could be an issue for the proposed secondary treatment (e.g. trickling filters), as the floatable would hang up on the media.

Odour Control

Any upgrades to the treatment should be designed to allow the installation of future odour control equipment if necessary. Because of its location across the Fraser River, the need for extensive odour control system will be established based on public input following construction of the new facilities. If it is deemed necessary to achieve a 5 dilution threshold at the property boundary, then significant odour control upgrades will be required.

Anaerobic Digestion

- Converting the mesophilic digesters to a thermophilic system will require significantly higher heating system components than a mesophilic system. The upgrade should include a heat recovery component to reclaim heat from the 55ºC sludge leaving the digester and raise the temperature of the cold raw sludge.

- The new sludge system to treat primary and secondary sludge should include a sludge blending tank and feed pumps to provide a uniform or homogenous sludge feed to digesters.

3.2.2 Lions Gate

A detailed site visit was carried out in the fall of 2003 to examine the current condition of the treatment equipment and unit processes. The purpose of the evaluation was not to prepare a maintenance and replacement program but to determine how the current primary treatment plant could be integrated with the proposed secondary treatment processes. The upgrades and improvements discussed in this section should be considered within the context of continued use of the existing plant infrastructure for the long-term future and ensuring that it can be successfully integrated with the proposed secondary treatment process.

Inlet Screens and Pump Station

The existing two screens have nearly adequate capacity (2 x 170 Ml/d = 340 Ml/d) to address any potential increase in peak wet weather flow of 356 Ml/d. A third similar screen needs to be installed in the available space to provide adequate redundancy. However problems of grit accumulation and screw conveyor overload will remain.

The existing pump station currently have adequate capacity to meet the current peak wet weather flow of 307 Ml/d but does not provide redundancy. It could be upgraded by installing a fifth pump with equivalent capacity of the existing largest unit to provide redundancy. Any amendment of the entry arrangements for the pumps would be difficult to implement since the suction piping is encased in the foundation of the caisson structure. Operation of the pumps must be maintained during any upgrade. The feasibility needs to be investigated and is beyond the scope of this report. The screening and pumping arrangements based on the above possible upgrading schemes would be far from ideal.

A detailed study is recommended to identify options for upgrading the existing inlet screen and pumping system. Among these is the use of Archimedes screw pumps to raise the flow to surface where screening and vortex degritting could be effectively operated.

Grit Removal

Grit accumulation, primarily in the digesters and wear of the centrifuges are problems consequent on the less than ideal performance of the existing aerated grit removal tanks. Should a further investigation exist for upgrading of the grit removal system, it could be achieved by the installation of two 8.5 m diameter vortex type grit removal tanks. These could be located in the space where digesters #1 and #2 presently stand. However, this space is also being considered for chemical dosing facilities Considering that build-out to secondary will not be carried out until 2030, the existing grit removal system will be over 70 years old at that time. A case could be made that this system will have reached the end of its life span by 2030 and should be replaced in conjunction with the build-out to secondary.

Primary Sedimentation Tank (PST)

Lengthening PST 3, 4, 5, 6, 7 and 8 as required, can best accommodate a required increase in capacity. Removing the end walls complete with launders and constructing new end walls and launders and extending the sludge and scum scraper mechanisms would lengthen the tanks. The existing pre-aeration and girt removal tanks will be retained whether or not new grit vortex removal tanks are installed. This cost of expansion work is estimated in Appendix.3 and 4.

Disinfection and Dechlorination

The existing chlorine contact tanks could remain in service until 2031. Thereafter a UV disinfection system could be considered as an option to replace the chlorination disinfection. If UV is installed, the existing system could be retained for the disinfection of flows in excess of 2 x ADWF which will receive only primary treatment. A UV system could be installed to replace the existing chlorination and dechlorination system.

Primary Sludge Gravity Thickener

Despite the sludge thickening function, the gravity thickener performs a useful function of sludge storage allowing semi-batch operation of the anaerobic digester. Unless interim secondary treatment is installed within a short period, a second gravity thickener should be installed to provide redundancy and the capacity needed for increasing loads prior to 2031.

Digesters

Digesters No. 1 and No. 2 should be kept for sludge storage or back up operation. These two digested could be demolished at a later date to make space available for upgrading of the plant headworks, or chemical dosing systems. Digester 1 is used as a sludge storage tank. It is needed as digesters 3 or 4 do not have fill and draw capacity. This issue needs to be addresses before Digester 1 can be demolished.

<u>Dewatering Systems</u>

The required capacity for the future, to at least 2046, can be achieved by installing an additional centrifuge in the space provided and increasing the operating times of the centrifuges.

<u>Odour Control</u>

The existing odour control facility on the sludge dewatering building will be retained. Any upgrades to the treatment should be designed to allow the installation of future odour control equipment if necessary. If it is deemed necessary to achieve a 5-dilution threshold at the property boundary, then significant odour control upgrades will be required since the plant is located close to build-up areas.

<u>Effluent Outfall and Diffusers</u>

The maximum hydraulic capacity is given as 341 Ml/d. The capacity of the existing system maybe inadequate under certain combinations of high sewage flows and high tides. A study is recommended to define the probability of failure.

3.3 ELECTRICAL AND POWER SUPPLY

3.3.1 Iona Island

The electrical systems of the Iona Island treatment plant are very well maintained, and a considerable amount of upgrading has been done recently. More upgrading, both to power and control system is planned over the next 3 to 5 years. Some weaknesses still exist, dating to the original construction, and these are identified in summary below.

Power Supply

The main energy requirements of the plant are supplied by the co-generation generators. There are currently 5 units, each rated 810 KW, and they are run in a configuration with 4 online and 1 standby. The generators are methane/natural gas fueled. These units were installed in 1998-99, replacing 6 older units and they are reported to be in good condition and working well. The plant was only connected to BC Hydro in approximately 1986, and operated as a stand alone, self-sufficient facility prior to that time. Highest peak load during the past year was 7493 KW (unverified data from CDAC System) which occurred December 14, 2004.

The B.C. Hydro power supply for the plant is at 12.47 KV via underground lines which come from South Vancouver and cross the Fraser River upstream of the plant. These lines are considered generally reliable, although one of them was out of service for three weeks, approximately two years ago. The plant presently has to import at least 500 KW of power, through the current agreement with B.C. Hydro, however it is reported that GVRD is negotiating with Hydro regarding rate structure and is considering the possibility of exporting power if it were available.

One reported weakness of the electrical system is that, although it was intended that the plant should have automatic load shedding in the case of the loss of the Hydro feeders if the generators were not able to carry the full load, this load shedding has never functioned correctly. There is further weakness in the co-generation control, which does not start and pick up load properly if all B.C. Hydro power is lost. At this point full manual intervention is required to bring the plant back on line. Based on the peak loading information, if a hydro outage were to occur at a time of high load, rapid effective load shedding would be required to prevent the plant from blacking out completely.

Essential Power

The main 12.47 KV switchgear has two 125 VDC battery banks for controlling the breakers. The original battery banks were replaced in 2000, along with the battery charger, and breakers. The Powerhouse programmable logic control (PLC) and some of the PLCs in the plant control system are powered by D.C., mainly as a result of harmonic problems in the plant electrical system. There is a separate Interruptible Power Supply (IPS) for lighting and other loads, which is good for approximately 20 to 30 minutes.

Backup Power

Backup power at the plant is provided by the co-generation generators. As more digesters are being installed for interim upgrades or build-out to secondary, the cogeneration plant should be expanded. Assuming that the IIWWPT fall under reliability Category 2 (Municipal Sewage Regulation) where there are no shellfish harvesting activities affected by the plant, backup power must be provided for the influent and effluent sewage pumping stations and for the primary clarifiers. Should the plant fall under reliability Category 3, back-up power would also be required for the primary clarifiers as well as influent and effluent pumping.

Distribution, Lighting, Motors and MCCs

The distribution in the plant is generally in good condition, as is the lighting. There is a capital program, beginning this year for the replacement of the aged motor control centers (MCCs). This replacement program also includes replacing the cables to the field equipment. Another improvement achieved by this replacement is the relocation of MCCs out of the basement areas (which would be most prone to flooding). Most underground cable on the site is in PVC duct and is reported to be in good condition.

There is 4,160 volts distribution to some major loads, and only 2 spare breaker locations in the 4,160 volts switchgear lineup.

Motors in general were observed and reported to be in good condition. The 6 main lift pumps equipped with variable speed D.C. motors have been regularly maintained. Some speed control drive logic has been replaced and all were running well at the time of this review.

Grounding

To the knowledge of the Electrical Supervisor, the plant grounding has never been tested, except that the ground grid was reviewed to some degree when the co-generation was built in 1998-99. Due to the age of the plant, and the nature of the river delta soils, it is very possible that the present grounding will require some replacement and upgrading.

Control Systems

Major upgrading and replacement of the plant automation system is already under way. The new CDACS system was partially operational at the time this review was conducted.

Data, Communication and Alarms

The telephone system is old and very basic. The replacement is included in the long-range plan, which may address this issue in the next 1 to 2 years. A fiber-optic backbone for the plant was installed by the GVRD I.T. department in 2003.

The plant was constructed before fire codes required installation of fire alarms. There is only one fire alarm system in the new annex and in the Generator Room. There are no sprinklers in the Plant. Addition of CCTV cameras for security and process monitoring is planned and a new main front gate, with the current standard monitoring and control, is to be installed in the current year.

<u>Issues Potentially Affecting the Upgrade</u>

- The present location of the incoming B.C. Hydro cables, which pass through the inner yard on the north side, may be fairly shallow. It might require relocation to accommodate the contemplated Plant upgrading.
- The existing transformers on site should be assessed to determine present condition, life expectancy, and suitability for the added load.
- There are transformers on both sides on the north side of the Operations Building which could be relocated if expansion of the building is contemplated.
- Layout of the main electrical room will not permit addition of new distribution and the room would either have to be expanded (which appears to be possible on the North side), or a second electrical room built at an appropriate location and sub-fed from present electrical room.
- Present harmonic problems should be assessed and corrected.
- The grounding systems in the Plant will likely require upgrading. This would only be determined following an in-depth review and testing.
- Fire alarms and fire suppression systems should be reviewed and will likely require major additions.

3.3.2 <u>Lions Gate</u>

Power supply for the plant is from the B.C. Hydro overhead lines, which are fed from the North Vancouver substation. The 12.47 KV lines enter the high voltage splitter and metering building, then feed 12.47 KV to the three plant 12.47 KV/480V transformers. These transformers vary in age. The newest is the Onan 1,000 KVA transformer (2003) which replaced the old 300 KVA unit. This feeds the administration building and primary sedimentation tanks No. 3 and No. 4. The distribution board has room for an additional 6 main breakers. The second, 750 KVA transformer feeds the sludge control building, and digester No.4, and primary sedimentation tanks No. 1, 2, 5, 6, 7, 8. Recently, the 1,000 KVA (1992) transformer in the dewatering building feeds the dewatering building and centrifuges.

The 600KW essential power generator was installed in 1991 and is in good condition. This feeds power to the plant essential service busbars through an automatic transfer switch. The essential grid load is normally fed by the 750 kVA transformer, via the genset transfer switch. The two VFD powered electrical raw sewage pumps, the maintenance shop, the digester gas compressors and fuel gas systems (for the gas driven raw sewage pumps), the hypochlorite and SO2 gas systems, administration building loads, and influent mechanical screens are on the essential service grid. The present load on the genset is 750 KVA. This leaves no capacity for future needs. The primary sedimentation tanks and grit tank blowers are included in the essential service grid.

The whole essential service grid setup at the plant is somewhat cumbersome, as there are duplicate power feeds to common buildings to accommodate the essential and non-essential power systems. To upgrade the plant, a more rational layout should be planned with the following processes powered from the essential service grid:

- Headworks screens
- Inlet pumps
- Gas systems supplying inlet pumps or co-generation system
- Primary sedimentation tanks
- Anaerobic digesters
- Chlorination system

MCC's and field control stations are undergoing a replacement program. In conjunction with this program the integration of the new plant CDAC/HMI control systems are being carried out. Several existing MCC areas require better ventilation for cooling, and there are no treated air systems in any of the MCC areas at present. The concern is that H2S gas and the seaside environment would cause accelerated degradation of electrical equipment, and especially instrumentation, PLCs and VFD's throughout the plant. These concerns need to be addressed in the overall plant long-term goals.

4 RECOMMENDED REPAIRS, UPGRADES AND COST ESTIMATES

4.1 IONA ISLAND

4.1.1 Ground and Foundation Improvements

Liquefaction of subsoil may cause instability and possible failure of the foreshore slopes around the IIWWTP. This may cause distress and possible damage to the various structures at this site. Some forms of ground improvement along the waterfront may be required to prevent ground and slope failure.

It is recommended that a 15 m wide area be densified to 13 to 14 m depth around existing IIWWTP facility. This densified berm would wrap around the entire facility. The purpose of this densified berm is to minimize the amount of liquefaction induced lateral movement of the ground. Note that liquefaction would still occur inside the non-densified area and below the existing structures. Also, flotation of in-ground tanks may occur. To prevent settlement of buildings and floatation of tanks, other forms of remediation such as soil anchors/mini-piles can be considered.

For any new site expansion, it is recommended that the footprint plus a 5 to 10 m wide envelope around the perimeter needs to be densified.

Soil anchors or steel pipe piles can be considered for providing resistance against uplift of buildings and tanks. Soil anchors can also be designed as mini-piles to provide additional axial compression capacity. The anchors can be installed within or around the perimeter of the building, provided that enough headroom for the machinery is available.

4.1.2 Major Plant Components and Equipment

a) Liquid Train System

Headworks

High velocities from the incoming siphons are causing excess stress on the bar screens, forcing screenable materials through bars and forcing grit into crevices. Modification of siphon discharge to reduce velocities, noise and related issues would be difficult and expensive. Any energy dissipation type structure would have to be located upstream of pump station, however, this would impact hydraulic elevations upstream. This could be impractical since there are also space limitations on site.

To reduce operating problems, another option would be to have the existing screens modified to trash racks (25 to 50 or 75 mm spacing) to protect the pumps and to have fine screens located after pumps. Any future conversion of influent screens to trash racks, should include new fine screens (6 mm spacing are recommended), located downstream of pump station.

One possible location for fine screens is at the end of the existing longitudinal grit channel. The location of the fine screens needs further study in relation to retention of the longitudinal grit tanks and distance of between the screens from the pump discharge. It is suggested to avoid placing too close to pump discharge to prevent wear/damage from grit in pump discharge.

The six influent pump casings are 40 years old and consideration should be given to replace those in order to extend the life of the station. However this can be considered to be an on-going maintenance item beyond the scope of this appendix.

The concrete wall surface of the longitudinal grit channel needs to be coated in order to correct the problem of exposed aggregate. This can also be considered an on-going maintenance item.

Instead of correcting individual problems with the current headworks as described above, another option is to install a new headworks building with new influent pumps, grit removal, fine screen and flow splitting chambers for improved flow distribution. A new headworks building would be located in the area south of the existing workshop. Considering that the headworks is the oldest portion of the plant, were built in 1962 and that the structure and many of the equipment will be over 50 years old when the build-out to secondary is construction in 2018-2020, it is recommended to provide new headworks when the plant is expanded to add secondary treatment. New headworks would solve the following problems: (1) energy dissipation of incoming sewage from the siphons, (2) old pump casings, (3) inadequacies in the grit removal system, (4) need for fine screening for trickling filters (5) flow splitting into the15 primary clarifiers (6) deterioration of concrete surface in grit channel.

<u>Influent Channel</u>

As indicated in Section 3.2.1, further investigation and hydraulic analysis is required to produce a protocol for sizing and placing specific window gate openings to improve flow splitting across the fifteen tanks.

The Hay & Company preliminary design report dated July 2002 had also suggested leveling of the effluent weirs. However, since it is proposed to use trickling filters (TF) or biological aerated filters (BAF) for secondary treatment, consideration should be given to convert the effluent weirs to submerged launder in order to minimize solids entrainment into the TF or BAF units. This matter should be examined at the time of detailed design.

b) Solids Train Components

In conjunction with upgrading the plant to secondary treatment in 2021, it is proposed to convert the anaerobic digesters from mesophilic to thermophilic mode. This is being considered in order to provide capability of producing Class A biosolids and thus increase the marketability of the biosolids. The need to produce a Class A product will have to be confirmed by long-term planning studies currently carried out by the GVRD. As indicated in Appendix 7, the proposed strategy for the plant upgrade is to provide future flexibility. Future digesters would be designed and sized for capability to be operated in the thermophilic or mesophilic mode. Similarly, in conjunction with the build-out to secondary, the existing digesters should be upgraded to provide the capability to operate in the thermophilic mode.

A study on thermophilic upgrade to the four existing digesters was carried out by Associated Engineering in November 1998. The estimated cost to provide heating for full thermophilic digestion of the four existing digesters was estimated at $1,904,000. When adjusted for inflation, this is estimated at $2,150,000.

c) Structural Repairs

As concluded in Section 2.2, with the following exceptions, the structures are generally adequate to accommodate seismic forces from the design earthquake. The following structures will suffer almost non-repairable damage and leaks from the design earthquake:

- Digesters 1 and 2
- Roofs of Stage II pre-aeration and sedimentation tank
- Precast panel anchored to digesters 1, 2, 3 and 4.

Also, some of the structures in the plant could suffer various types of damage due to differential ground movement as a result of liquefaction. Unless liquefaction mitigation measures are implemented, this could cause damage to the structure. Corrective measures to address this issue are summarized in Section 4.1.1.

A walk-through and visual inspection was carried out to verify the condition of the concrete tank and structure. Most of the treatment plant buildings, tanks and pipe galleries appear to be in good condition. Some repairs will be required as follows:

- Provide repair mortar patches to spalled concrete wall of Tank No. 5, and protect reinforcing steel from corrosion damage.
- Replace or trim buckled steel plate covers to relieve build-up stresses.
- Replace expansion/control joint sealant as required.
- Check and seal crack in slab at effluent pump station.

- Monitor H_2S attack on wall and slab in effluent channel. Replace floor slab if reinforcing steel is exposed.

4.1.3 Options for Improving Flow Separation to Primary Sedimentation tanks

As indicated in Section 4.1.2 above, an interim solution to improve the flow separation to the fifteen primary sedimentation tanks is to properly size the openings between the influent channel and the pre-aeration tanks, by adjusting the size of the opening of the metal plates that are inserted in the concrete openings. Other more elaborate options have also been identified and would be implemented as part of the build-out to secondary treatment:

1. Option 1 - Flow Splitting Chambers

   With this option, the flow to the primary sedimentation tanks would be split with overflow weir located in several splitting chambers. Flow splitting chamber # 1 would have four outlets overflow weirs. From each of the four outlets, the flow would be directed to four splitting boxes each would have 3 or 4 overflow weirs to the individual primary sedimentation tanks.

   The implementation of two levels of flow splitting chambers would require more head that is currently available with the present headworks arrangement. This option would essentially require new headworks with larger pumps, new grit removal (e.g. vortex type grit removal). The proposed new fine screens could also be incorporated into the upgrade. Since it is proposed to install new headworks, flow splitting chambers could be incorporated with the headworks.

2. Option 2 - Outlet Control and Submerged Launders

   With this option, the existing launders and outlet weirs in the fifteen primary sedimentation tanks are replaced with submerged launders. For each primary sedimentation tank, an effluent pipe would connect the submerged effluent launders to a new effluent channel located parallel to the existing effluent channel. A flow metering device and an automated control gate would be installed on each of the fifteen discharge pipes. A new effluent channel would be required to provide the space for the new discharge pipe from each primary sedimentation tank and to compensate for the additional head losses generated by the new piping and equipment. Flow measurements of the incoming flow (degritted primary influent) would also be needed in order to control the flow out of each of the fifteen primary sedimentation tanks. The cost of installing submerged effluent launders including controls and automated valves is estimated at about $330,000 per primary clarifier for a total of $5 million for 15 clarifiers.

4.1.4 Summary of Preliminary Cost Estimates

This section includes preliminary cost estimates for the recommended upgrades for Iona Island as identified in Section 4.1.1 to 4.1.3 above. The following cost estimate does not include on-going equipment repairs, maintenance and replacement. These cost estimates are provided only for the items that will be needed for upgrading the existing primary treatment plant for the following reasons:

1. To integrate the existing primary plant with the proposed secondary plant.

2. To ensure that the treatment facility can be operated satisfactorily for the next 50 years.

However, in some cases it is difficult to differentiate upgrading cost with on-going maintenance cost. Table 4.1 summarizes the proposed plant upgrades and indicates if the repair/upgrades are part of on-going maintenance or required to integrate the existing plant with the proposed secondary plant and extend the life of the plant. As indicated in Table 4.1, the costs of some of these upgrades are included in the cost estimates for the built-out to secondary found in Appendix 4 and 10.

TABLE 4.1
SUMMARY OF UPGRADES TO IONA ISLAND PRIMARY TREATMENT PLANT

Description of Upgrades	Estimated Cost	Remarks
1. Ground densification around existing primary plant – 15 m wide area to a depth 14 m	$1,200,000	Amount included in cost estimates in Appendix 4 and 10 for build-out to secondary
2. Soil anchors to provide resistance against uplift of existing building and tanks	$8,880,000	Amount included in cost estimates in Appendix 4 and 10 for build-out to secondary
3. Hydraulic analysis for sizing and placing influent gates into the primary sedimentation tanks.	$85,000	Amount included in cost estimates in Appendix 10 for interim upgrades
4. New headworks including influent pumping, coarse screening, grit removal, fine screens, and splitting chambers	$55 million (allowance)	Amount included in cost estimates in Appendix 10 for build-out to secondary.
5. Submerged launders in primary sedimentation tanks and outlet control	$5 million	Amount included in cost estimates in Appendix 10 for build-out to secondary.
6. Thermophilic upgrade to the four existing digesters as per recommendation of Associated Engineering report date Nov 98 – cost estimate indexed	$2,150,000	Amount included in cost estimates in Appendix 10 for build-out to secondary
7. Replacement of digesters 1 and 2 to provide seismic protection for a 1:475 year design earthquake	$6.5 million	Because of the high cost of replacing digester, this amount is not included in the cost estimate in Appendix 10
8. Bracing of precast panels anchored to digesters 1 to 4 to prevent collapse in a 1:475 year design earthquake	$500,000 (allowance)	Amount included in cost estimates in Appendix 10 for build-out to secondary
9. Roof of Stage 2 pre-aeration and primary sedimentation tanks to prevent collapse in a 1:475 year design earthquake.	$500,000 (allowance)	Amount included in cost estimates in Appendix 10 for build-out to secondary
10. Miscellaneous concrete repairs		It is assumed that repairs will be carried out by GVRD as part of on-going maintenance. **Not included** in cost estimates in Appendix 10

4.2 LIONS GATE

4.2.1 Ground and Foundation Improvements

Liquefaction induced lateral spreading of the ground towards Burrard Inlet may cause distress within the existing structures. Some forms of ground improvement along the waterfront may be required to prevent lateral spreading of the ground.

For a preliminary design, it is recommended that a 15 m wide densified berm along the length of the south and southwest boundary of the site facing the Burrard Inlet be considered. Densification would have to be carried out from the existing grade to a depth of 15 m. Densification using vibro-floation or replacement can be considered. Presence of possible boulders within the Unit 2 soil layer may prevent penetration of the vibrating probe. To assess this problem, a densification test section with 15 m x 15 m in plan area, is recommended.

The purpose of the densified berm is to reduce the amount of liquefaction induced lateral movement of the ground towards the Burrard Inlet. Note that liquefaction would still occur inside the non-densified area of the site and below the existing structures. Therefore, post-liquefaction settlement, in the order of 200 to 300 mm would still be expected. Also, flotation of in-ground tanks may occur. To prevent settlement of buildings and flotation of tanks, other forms of remediation such as soil anchors/mini-piles can be considered.

Soil anchors or steel pipe piles can be considered for providing resistance against uplift of buildings and tanks. Soil anchors can also be designed as mini-piles to provide additional axial compression capacity. These anchors can be installed within or around the perimeter of the building, provided that enough headroom for the machinery is available.

4.2.2 Major Plant Structures and Equipment

a) Liquid Train Systems

Headworks

The present headworks' coarse screening system suffers from grit overload during high flow events. The flow distribution to the screens is also not ideal in that the flow has to turn through two (2) 90 degree channel bends, exit through another 90 degree bend, enter the old screening channels, and then the buried inlet piping feeding to the suction of the four pumps. The capacity of all four existing pumping units is used at present during these high flow events. The layout and accessibly of the pumping room can be considered cramped, when compared with newer GVRD plants. Further, the pumps are now old, as are the two gas-powered motors, which drive the two raw sewage pumps.

The location of the old VFD electrical equipment and MCC in the hot gas powered motor room area is not ideal from a cooling viewpoint. If these pumps and motors were all to be replaced with new pumps and VFDs, a new MCC room would be required.

The efficiency of using gas-powered motors for driving pumps through a gearbox versus installing a cogeneration power plant and conventional VFD pumps should be investigated. A cogeneration system review of the plant is understood to be in hand.

Rapid wear to the raw sewage pump impellers is understood to be caused by grit. These damages to the impellers require frequent replacements. There are also some cavitation problems associated with the present intakes.

In the light of the above, it is recommended that ideally the existing headworks and pump station be abandoned, and that a new headworks complete with grit removal, screens and influent pumping station and MCC be incorporated into the proposed long term plant upgrade.

New piping (or a channel) feeding from the new raw sewage pumps would be designed for the seismic event. This would correct this deficiency in the existing pump discharges.

<u>Influent Channel & Pre-aeration Tanks</u>

The new pre-aeration tank influent channel control gates seem to have largely addressed the unequal flow split to the primary sedimentation tanks. The roll aeration grit systems in these tanks can be abandoned once a new grit removal system has been constructed and the equipment been removed. The aeration headers should remain.

Concrete repairs to correct surface erosion and H2S attack will be required as part of ongoing maintenance. Structurally these tanks are sound, except the roofs will need to be upgraded or removed because of seismic concerns.

It is not thought to be cost effective to retrofit the pre-aeration tanks to achieve longer primary sedimentation tanks. The costs to retrofit sludge hoppers, piping and collector mechanisms are prohibitive, for a minimal additional benefit.

The mechanical systems are largely well maintained and repaired. Aside from seismic improvements, little maintenance is required. The existing grit auger, pump and cyclone systems will require ongoing component replacement and renewal, if they are kept commissioning.

Primary Sedimentation Tank

The roof structures above the PSTs need some seismic upgrades or to be removed. Concrete repairs to correct surface erosion and H2S attack will be required as part of ongoing maintenance. Structurally these PSTs are sound.

The scum removal system is in two parts. One collects scum at the outlet end of the tanks via the surface longitudinal collector flights, the other uses spray bars to drive the scum towards the front of the tanks. Neither scum removal system is automated. The primary sedimentation tanks should be upgraded with a new scum collection system, with the appropriate changes to the longitudinal collectors.

Where additional primary sedimentation tank capacity is required, consideration should be given to extend the shorter primary sedimentation tanks to match the larger tanks. Additional primary capacity can be provided by the expansion of PST surface area.

The collector drive mechanisms are undergoing a replacement program. The sludge pumping system is in good condition, but plugging is a common problem in the discharge header lines. Seismic improvements to the piping systems in key areas should be considered (i.e. at tank exits, pipe gallery corners, and across tank/gallery expansion joints).

Chlorine Contact Tanks

These tanks require some seismic improvements and the leaking water stops should be repaired. There is minimal surface restoration of the concrete required. Continuing the use of chlorine disinfection for the plant is proposed.

The existing SO_2 dosing system and flow measurement building at the end of the chlorine contact tank will need some minor roofing and corrosion related repairs. The possible elimination of the ton SO_2 cylinders and the replacement with a liquid SO_2 system is likely. In the short term, this building and SO_2 gas system is adequate.

b) Solids Train Components

Sludge Thickener

This thickener is simple to operate and provides the 4-5% concentration thickened primary solids to the digesters. The tank also acts as sludge storage reservoir. However, this tank has settled unevenly. If the long-term settlement concerns for this tank can be addressed then it can serve as a backup thickener with the required redundancy. Otherwise, it should be removed and replaced with a new thickener system. In either case, at least one additional new thickener is required to add redundancy to this system.

If the long-term odour control goal of 5 DT at the property line is desirable then an enclosed thickener system is required. In this case the tank should be abandoned or kept for emergency use only. Any new thickener system should be equipped with odour control.

Digesters

Existing digesters No. 1, N0. 2 and the associated mechanical works should be demolished. Digester 3 & 4 should be kept for the interim and then be replaced with new structures that can meet the desired seismic standards. The existing digester piping systems and digester gas piping systems are under constant replacement programs and the asbestos insulation is also being removed. These piping systems are of various ages and conditions. With the ongoing program of replacement the systems will last for the interim time frame.

The digester gas high-pressure compressors are now old and may require replacement.

The digester building NO.3 MCC has recently been upgraded.

Digester No. 4 has undergone a retrofit in 2000-2001 and digester No. 3 is presently under a retrofit program. The floating roof is being replaced with a fixed roof, and the in-tank mixers and concrete are all being renovated.

Dewatering Systems

The dewatering facility and equipment was built in 1991. Structurally this building and the equipment are in good condition. There is some seismic improvement work required in the underground pipe gallery to address flexibility across the new and old piping interface.

The centrifuges are currently in good condition. Grit which causes wears and tears on the centrifuges is an ongoing maintenance issue.

Odour control, which was added after the initial construction is marginal on this building and a high capacity system will be required in the future.

Truck access in and out of the sludge hopper loading area is not ideal but is sufficient. If the site can be reworked to allow for drive through, then this should be incorporated into the design. The use of a loader to maneuver the bins in and out is currently workable, but will become onerous once the sludge volumes increase by a factor of more than two.

c) Ancillary Systems

Water

The plant is presently fed by a single 150 mm water main from the West Vancouver distribution system. The available fire flows are limited. A new 300 mm line is proposed, or possibly a well for fire protection and perhaps for all other plant non-potable water use (hose stations etc.).

Air

The compressed air is used in limited amounts at present. The plant compressor is new and is in good condition. The location in the genset room is not ideal, but it is readily serviceable.

Plant Automation

At present the plant is under going a controls system upgrade. This is scheduled to be completed in the next two years. Ongoing plant expansions will be automated to the GVRD's standard.

Administration and Laboratory Building

The seismic review indicates that this building is structurally adequate provided some improvements to the fusing of glass panels are made. The building is adequate for its present staffing levels. In the future more offices, meeting area, and a larger laboratory will be required. It is proposed that the existing laboratory area be converted into new offices or a control room area, and a new laboratory area be added on. The women's and handicapped person's washroom facilities also require upgrading. Parking area adjacent to this building also needs to be expanded.

Machine Shop and Supplies Buildings

Both of these structures are old and inadequate. They should be demolished and a new workshop complete with overhead cranes, machine shop, welding area, painting area, and storage facilities can be provided.

Odour Control

The plant presently experiences a number of odour complaints from the surrounding areas. The closest residents are only about 100 m away. The plant presently has odour treatment only for the dewatering building. In order to meet a 5DT at the property line goal, many of the solids train processes and buildings will require odour containment and treatment.

Other processes which may require odour control include:

- Primary sedimentation tanks
- Headworks
- Secondary treatment (aeration systems)

4.2.3 Preliminary Cost Estimates

This section includes preliminary cost estimates for securing the capacity of the existing plant, where it is feasible to make these on the basis of available information. The cost estimate does not include ongoing equipment repairs or maintenance and replacement of equipment, structures or services. The estimate does not make provision for upgrading of the capacity to meet growth in flow or load beyond the current capacity. Those costs are included in the Appendix 3. Costs for upgrades included in Appendix 4 have also been included in the Appendix 10 report.

The summary of potential upgrades to the Lions Gate primary plant together, with an indication of level of knowledge and where costs are defined, is included in Table 4.2.

TABLE 4.2
SUMMARY OF POTENTIAL UPGRADES TO LIONS GATE TREATMENT PLANT

Description of Upgrades	Estimated Cost	Remarks
1. Ground densification of a 15 m wide berm along the South and South-west boundaries to a depth of 15 m	\$1,680,000	Amount included in Appendix 3 and
2. Soil anchors to provide resistance against uplift of existing structures No specific provisions has been included to protect expansion joints.	\$1,480,000	Amount included in Appendix 3 and
3. Inlet screens and pump station – extent or method of upgrade not determined.	\$10 million (allowance)	Study recommended to identify needs and methods for achieving more suitable configuration, possible increase in capacity and adequate redundancy. Existing screen and pump station will have probably reached its lifespan when the build-out to secondary takes place in 2030
4. Grit removal –	\$4 million (allowance)	Business case study recommended to establish need for improved grit removal. Existing grit removal system will have probably reached its lifespan when the build-out to secondary takes place in 2030
5. Primary sedimentation tanks • No upgrade needed for present capacity. • Seismic upgrade of roof structures.	Nil \$200,000	Upgrade for increased capacity included in Appendices 3 and 4 Amount included in Appendices 3, 4 and 10
6. Disinfection and dechlorination • No upgrade needed for present capacity.	Nil	Upgrade for increased capacity included in Appendix 3 and
7. Sludge Thickener • No upgrade needed for present capacity.	Nil	Upgrade for increased capacity included in Appendix 3 and
8. Digesters • No upgrade needed for present capacity.	Nil	Upgrade for increased capacity included in Appendix 3 and
9. Dewatering system • No upgrade needed for present capacity.	Nil	Upgrade for increased capacity included in Appendix 3 and
10. Odour control • The extent of required upgrading to be confirmed.	\$2 to \$4 million	
11. Effluent Outfall and Diffusers	N/A	Study recommended assessing the need for additional capacity.

5 REFERENCES

Dayton & Knight 1998, Iona Island Wastewater Treatment Plant Grit Removal Study and System Upgrade Predesign.

Hay & Company 2002, Preliminary Design of Flow Equalization Infrastructure at Iona Island Wastewater Treatment Plant.

Associated Engineering 1998, Thermophilic Upgrade to Digesters at the Iona Island Wastewater treatment Plant.

APPENDIX A: GVRD SEISMIC DESIGN CRITERIA

GREATER VANCOUVER REGIONAL DISTRICT	Page -	Approved
ENGINEERING STANDARDS	Item -	
SEISMIC DESIGN CRITERIA	Section -	Date Dec. 17, 2002
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	SUPERSEDES	

The Greater Vancouver Regional District owns, operates and maintains the large-scale systems for storing, treating, and supplying potable water, for collecting, conveying, and treating wastewater and for managing and disposal of solid waste relied on by the region's municipalities, residents, businesses, industries and institutions. As such it provides an essential service. These systems are expected to survive a moderate earthquake with minimal disruption and a severe earthquake with manageable disruptions. A committee comprised of representatives from both the Engineering & Construction and Operations & Maintenance Departments was formed to standardise the seismic criteria to which all new and remedial works are to be designed. The Seismic Design Criteria have been established based on the District's post disaster operating objectives and review of the public safety consequences of failure for each facility.

The seismic design criteria are provided as a guide for the designer and are to be considered as desired levels of seismic resistance rather than absolute minimum requirements. When designing new facilities or upgrading existing facilities, the cost of meeting the seismic design criteria must be weighed against the importance of the facility for system operation (i.e. redundancy in the system), and any life safety issues associated with damage to the facility. The table below presents the facility class and the expected performance levels for the NBCC and MCE levels of earthquake.

Table Indicating Expected Performance of GVRD Water, Sewer and Solid Waste Infrastructure when subjected to the National Building Code of Canada Earthquake (NBCC) and the Maximum Credible Earthquake (MCE)

FACILITY	NBCC 1:475 Return Period	MCE 1:10000 Return Period
Dams	IO	LS
Water Treatment Plants	IO	LS
Wastewater Treatment Plants	LS	-
Primary Water Disinfection Facilities	IO	LS
Secondary Water Disinfection Facilities	LS	-
Pump Stations – Level 1(2)	IO	LS
Pump Stations – Level 2(2)	LS	-
Pipelines(3) – Level 1(2)	IO(1)	LS(1)
Pipelines(3) – Level 2(2)	LS(1)	-

Segmental Pipe[4]	LS[1]	-
Chambers and Portals - Level 1[2]	IO	LS
Chambers and Portals – Level 2[2]	LS	-
Reservoirs – Level 1[2]	IO	LS
Reservoirs – Level 2[2]	LS	-
Ancillary Structures	CP	-
Water Transfer Stations	LS	-
Municipal Landfills	LS	-
Burnaby Incinerator	LS	-

(1) - See the section on the performance levels and preliminary design procedures for buried pipe.
(2) - Level 1 and Level 2 facilities are described in the definition section.
(3) - High pressure continuously bonded pipes such as welded steel, polyethylene or flanged steel.
(4) - Consult with pipe manufacturer on the recommended allowable deflection at the joint.

Definitions

NBCC The National Building Code of Canada provides an Earthquake spectrum, which can be used for analytical modelling, and a minimum base shear equation in section 4.1.9.1.(4). The NBCC earthquake has a probability of occurrence of approximately once in 475 years.

MCE The "Maximum Credible Earthquake" is site specific and has a probability of occurrence of approximately once in 10,000 years. The MCE design level is only appropriate for those facilities that present a high hazard to properties and life. MCE design spectra can be established using either a deterministic or probabilistic approach.

Level 1 Facility is critical to system operations, has little or no redundancy, and may result in substantially reduced service for an extended period (months) if failure occurs. This level includes pipelines for marine crossings.

Level 2 Facility is important to the system operations, however some level of redundancy exists, service will only be marginally impacted for a limited period (days to weeks) if failure occurs. For example many pump stations are only required for summer service and may therefore warrant a lower importance rating.

Expected performance Levels for Buildings as defined in the FEMA 273 Guidelines and GVRD Expected Performance Levels for Structures

Immediate Occupancy (IO) Performance Level

FEMA
Structural performance level IO, Immediate Occupancy, refers to the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor repairs may be required, these would generally not be required prior to reoccupancy.

GVRD
Concrete water retaining structures such as dams and reservoir walls should exhibit near elastic response with no damage. Earth fill dams may experience minor surface cracking but should not allow the passage of water. Superstructures such as reservoir roofs may exhibit only minor damage such that repairs can be completed without interrupting facility operation, or that the structure can remain operational for an extended period of time prior to completing the repairs.

Life Safety (LS) Performance Level

FEMA
Structural performance level LS, Life Safety, refers to the post-earthquake damage state in which damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons it may not be practical. While the damaged structure is not an immediate collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupancy. This type of performance would be similar to that expected of post-disaster structures in the NBCC. This performance level could be achieved by several methods. One method is to include an importance factor (I) of 1.5, thus increasing the design forces by 50%. An alternative means of achieving this performance level would be to increase the ductility of the structure rather than the strength. This would involve detailing the structure to a higher level of ductility (R value) than the code requires for the chosen lateral system.

GVRD
Water retaining structures may experience distress, cracking and minor leakage but must remain operational at full capacity. Superstructures may experience significant but repairable damage but should not be in a condition of potential collapse. Repair of both

the water retaining structures and the superstructure may require the facility to be out of service for up to 2 months during the non-peak demand season.

Collapse Prevention (CP) Performance Level
FEMA

Structural performance level CP, Collapse Prevention, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure, and, to a more limited extent, degradation in vertical load carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse.

GVRD

Water retaining structures will experience distress, cracking and moderate leakage but should be operable at less than full capacity after a thorough inspection (less than 3 days down time). Superstructures will exhibit significant damage (cracking and spalling of concrete) but will not collapse. The superstructure may be repairable, but if damage or deflections are excessive the structure would have to be replaced. Repair of the water retaining structure and repair/replacement of the superstructure may require the facility to be out of service for up to 6 months during the non-peak demand season.

<u>Performance Levels and Preliminary Design Procedures for Continuous Welded Buried Pipe</u>

The seismic design of buried pipes is a recent field of study. Many papers and articles on this topic have been published over the last 5 years. This research has been undertaken as a result of damage assessments from earthquakes in both the USA and Japan. Buried pipelines can be damaged either by transient seismic wave propagation or by permanent ground deformation. For the analysis and design of buried pipelines, the effects of seismic wave propagation are typically characterized by the induced ground strain and curvature, which, in turn, can be characterized by the peak ground motion parameters (acceleration and velocity) as well as the appropriate propagation velocity. Simple solutions are available (e.g. Newmark, 1967) to estimate these ground strains. However, observations from past earthquakes suggest that damage rates of buried pipeline by transient seismic wave propagation are low. It is often found that the effects of transient seismic wave propagation do not govern preliminary pipeline design.

Although permanent ground deformation hazards are usually limited to small regions within the pipeline network, their potential for damage is very high since they impose large deformation on pipelines. Permanent ground deformation includes surface faulting, lateral spreading due to liquefaction, and landsliding. With input from the geotechnical engineer, regions with surface faulting and slope instability should be avoided during preliminary alignment selection for the pipeline. However, in the Lower Mainland, it is often not feasible to avoid areas with the potential for lateral spreading due to liquefaction (e.g. near river banks and the Fraser delta). For preliminary design, the following simple procedure should be used to assess the effects of permanent ground deformation due to liquefaction.

It is assumed that pipe deflections caused by ground deformation will approximate a sinusoidal wave. The magnitude of the lateral displacement, and the length over which that displacement occurs, must be provided by a geotechnical engineer. The following symbols are used to define the parameters used in the total stress calculations.

Symbols

σ_b	=	bending – axial tensile stress caused by the lateral deflection
υ	=	shear – stress due to the applied load, normal to the pipe axis
σ_1	=	axial – longitudinal tensile stress due to the internal pressure
σ_2	=	Hoop – tangential stress normal to the pipe axis due to the internal pressure
σ_T	=	temperature stress – negligible in the GVRD area for buried application
σ	=	total stress

The total stress in the pipe wall is determined by combining the orthogonal stresses using the Root Sum Square (RSS) method as follows:

$$\sigma = \sqrt{(\sigma_1 + \sigma_b)^2 + {\sigma_2}^2 + \upsilon^2}$$

The performance levels for pipe are then defined as a ratio of the total stress calculated divided by the yield stress (F_y) of the pipe material as follows:

Immediate Occupancy (IO) $\Rightarrow$ $\sigma \leq F_y$

Life Safety (LS) $\Rightarrow$ $\sigma \leq 1.15\ F_y$

Collapse Prevention (CP) $\Rightarrow$ $\sigma \leq 1.30\ F_y$

The above equations will be calculated using the predicted lateral deflections contained in the preliminary geotechnical investigation. In cases where very soft or liquifiable soils exist, an economic analysis should be performed to determine which remedial method is most cost effective. It should also be noted that pipe strength and ductility may be governed by the pipe joints and care must be taken to select the correct type of joint for each application.

References

Cleveland and Seymour Falls Dams, Stability Assessment - Phase 3, Design Basis Memorandum, by KCCL, in association with BCHIL (1991).

Seymour Falls Dam - Seismic Hazard Review Update - BCHIL - 1998.

Seismic Evaluation and retrofit Design - Vancouver Heights Reservoir - Evaluation Report - Sandwell - 1993

Canadian Dam Association (1999)- Dam Safety Guidelines- January 1999.

"Steel Stress due to Pipe Deflection"- Greater Vancouver Regional District, Department of Engineering Standards and Instructions, Section 02210- June 2000.

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 9
Geotechnical Assessment

FINAL REPORT

Prepared for

Greater Vancouver Regional District

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August 2005

117-00018

Trow

FACILITY PLANS FOR IONA ISLAND AND LIONSGATE WASTEWATER TREATMENT PLANT

GEOTECHNICAL ASSESSMENT

DECEMBER 20, 2004

Prepared for:

STANTEC CONSULTING LTD.

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EXECUTIVE SUMMARY

A preliminary Geotechnical assessment was carried out for the Greater Vancouver Sewerage & Drainage District's Facility Plans project for Iona Island and Lions Gate Wastewater Treatment Plants. The assessment includes review of the subsoil conditions, foundations, seismicity and potential rise in sea level. Preliminary assessment and recommendations for the proposed pipeline routing from Lions Gate wastewater treatment plant to Iona Island wastewater treatment plant are also provided. A brief summary of the assessment and recommendations are given below.

Iona Island Wastewater Treatment Plant (IIWWTP)
The IIWWTP site is located in the central portion of Iona Island, which is located to the south of Vancouver. The North Arm of the Fraser River separates Iona Island and Vancouver.

Subsoils at the IIWWTP site consist of deltaic deposits from the Fraser River, comprising of unconsolidated silts, sands and silty clays, more than 100 metres in thickness overlying dense to very dense pleistocene glacial soils. The site has been raised using approximately 4.5 m thick river sand fill prior to construction of the existing structures.

The IIWWTP site has been preloaded in several phases prior to construction of the existing facilities. Major portions of the site have been preloaded prior to the original construction over a 2-year period from March 1959 to May 1961. It is understood that preloads with 2 to 6 month duration were used for the construction of the various additions to the earlier structures. A review of the preload and settlement history indicates that with an 8.5 m high preload, maximum settlement of 1.82 m was observed over a 2 year duration. Post construction settlement as high as 0.7 m was measured over 35 years. Preliminary recommendations for future preloading and set-back distance are given.

It is understood that for the seismic upgrading of the existing structures recommendations given in the NBCC 1995 (475 year return period earthquake motion) are to be used. Significantly thick zones of loose sands below the surficial fill zone are expected to liquefy due to the 475 year return period earthquake motion.

Liquefaction would likely cause deformation of the ground, dykes, building foundations and floatation of lightly loaded in-ground tanks. Recommendations for ground improvement and foundation upgrade options are given to mitigate liquefaction induced ground and foundation deformations.

Lions Gate Wastewater Treatment Plant (LGWWTP)
Subsoils at the LGWWTP site consist of: a maximum 1.8 m of FILL comprising sand and gravel with pieces of wood, debris, and organics; a 13 to 15 m thick layer of SAND and GRAVEL with some cobbles and boulders; a 25 to 35 m thick layer of silty SAND with some gravel; a 20 to 40 m thick very dense glacial till overlying claystone bedrock.

For the design 1:475 year return period earthquake motion, potential liquefiable zones at the LGWWTP site are expected to be scattered sporadically throughout the site, with some local zones of significant liquefaction. Earthquake shaking together with subsoil liquefaction would likely cause ground settlement and movement towards Burrard Inlet. Recommendations for ground improvement

to prevent lateral spreading are presented. Recommendations for foundation upgrade, such as soil anchors, minipiles, steel pipe piles and so forth are also given.

Seismic design parameters and lateral earth pressure on basement walls are provided for soil-structure interaction analyses.

A preliminary pipeline route assessment has also been carried out.

The assessment results given in this report are provided for planning purpose only. Detailed design and analysis would be needed for the final design. The detailed analysis would require subsoil data, which would have to be obtained from site-specific drilling methods such as Cone Penetration Tests, Standard Penetration Tests and/or other equivalent methods. The analysis would include liquefaction assessment, estimation of seismically induced ground deformation, foundation bearing capacity, settlement and so forth.

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ATTACHMENTS

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FACILITY PLANS FOR IONA ISLAND AND LIONS GATE WASTEWATER TREATMENT PLANTS

GEOTECHNICAL ASSESSMENT

1 INTRODUCTION

This report presents the findings of the geotechnical assessment and preliminary design recommendations for the upgrading of the Iona Island and Lions Gate Wastewater Treatment Plants (IIWWTP and LGWWTP). The purpose of this assessment is to assist the design team for the preparation of facility plans for the above noted plants in accordance with the approved Liquid Waste Management Plan. Facility plans are required to identify the options available, and their associated costs, to meet all required objectives and Corporation's needs in the interim 20 to 30 year planning horizon before upgrading to full secondary treatment becomes a requirement.

Stantec Consulting Ltd. (Stantec) is the prime consultant to GVSDD. The Geotechnical Assessment work was undertaken in accordance with our proposal dated July 17, 2003. Scope of the Geotechnical Assessment work is outlined in the Stantec Letter dated July 16, 2003. Trow Associates Inc. (Trow) is a sub-consultant to Stantec. Structural Engineering assessment is provided by Stantec for the IIWWTP and by Dayton & Knight Ltd. for the LGWWTP.

The scope of Geotechnical Assessment work, briefly, is:
- Gathering and consolidating existing related geotechnical information and data;
- For the IIWWTP and LGWWTP, identify and evaluate potential foundation and construction aspects with respect to proposed interim and long-term upgrades;
- To provide supplementary geotechnical advice for new locations where LGWWTP would be relocated or new facility developments considered;
- To provide a preliminary pipeline route assessment (of the proposed routes by GVSDD) from North Vancouver to Iona Island.

Section 2 of this report presents the assessment of the IIWWTP. Geotechnical assessment of the LGWWTP is presented in Section 3. A brief discussion on the effects of potential rise in sea level is presented in Section 4. Section 5 provides a preliminary pipeline route assessment from North Vancouver to Iona Island.

Attached to this report are:
1) Key Plans showing the location of the IIWWTP and LGWWTP;
2) Aerial Photographs of the existing IIWWTP and LGWWTP;
3) Preload and settlement contours of the IIWWTP and;
4) A diagram showing lateral earth pressure on basement walls.

No drilling was carried out as part of this assessment. Published data from previous investigations was utilized for the assessment.

2 IONA ISLAND WASTEWATER TREATMENT PLANT (IIWWTP)

The Iona Island Wastewater Treatment Plant (IIWWTP) was opened in 1963 and has been expanded six times to increase capacity and for upgrades. The discussions given in the following sections are based on review of the following documents:

- Technical paper by Charles F. Ripley (1995), Preloading thick compressible subsoils: a case history, Canadian Geotechnical Journal, Vol.32, pp.465-480.
- Klohn Crippen report, IIWWTP Cogeneration system replacement, dated Aug. 14, 1996
- GVRD report, Lagoon dyke access road upgrade, dated Aug. 31, 2000;
- GVRD Engineering Standard – Seismic Design Criteria, report dated Dec. 17, 2002.

The various stages of major developments, between 1959 and 2002 are summarized in Table 1. Note that this table presents the information made available to us during the writing of this report. Further details may be found in GVRD files.

Table 1. Major developments at IIWWTP site between 1959 and 2002

Year	Development
1959 - 1961	Preloading for the original IIWWTP
1959	Sedimentation tanks 1 to 5; Digesters 1 and 2; Pump building; Grit chamber 1 to 6; Maintenance building; Power and Administration building
1972	Sedimentation tanks 6 to 10; Thickener No. 1 (demolished since 1985) Scum removal system;
1978	Digesters 3 and 4; Thickener No. 2
1981	Sedimentation tanks 11 to 13
1985	Sedimentation tanks 14 and 15; New thickener No. 1
1986	Effluent pump station; Outfall piping
1995	Screen? building
1997	Seismic upgrading of pump building
1998	Seismic upgrading of power and administration building
1999	Roof replacement for digester No. 3

2.1 SITE AND SUBSOIL CONDITIONS – IIWWTP

The IIWWTP site is located in the central portion of Iona Island, which is located to the south of Vancouver. The North Arm of the Fraser River separates Iona Island and Vancouver as shown in Dwg. 1.

Subsoils at this site consist of deltaic deposits from the Fraser River, comprising of unconsolidated silts, sands and silty clays, more than 100 metres in thickness overlying dense to very dense pleistocene glacial soils.

Original grade elevation was 1.46 m ± Geodetic prior to the start of 1960's works. The existing grade was raised to El. 4.0 m ± by using dredged river sand fill.

Summary of sub-soil profile based on the available test hole data is given in Table 2.

Table 2. Generalized sub-soil profile, IIWWTP

Soil units	Approximate Thickness, (m)	Description
Unit 1	4.5, (El. 4 to –0.5 m)	FILL - compact to dense river sand
Unit 2	7.5, (El. –0.5 to –8.0 m)	loose to compact interlayered SILT and SAND
Unit 3	3.5, (El. –8.0 to –11.5 m)	compact river SAND
Unit 4	>100, (El. < -106 m)	compressible SILT

Groundwater table elevation obtained from the various drill holes varied from 0.7 m to 1.2 m and is expected to vary seasonally and with the amount of precipitation and tidal levels.

Note that the groundwater and sub-soil profile conditions described above were derived from the available reports and test hole logs for this site. These are conditions at the test hole locations on the day of measurements only and may not be representative across the site.

For detailed design of any future structures additional drilling would be required. This additional drilling would provide information on changed soil conditions at this site due to past preload settlement, and information for seismic analysis and design.

2.2 PRELOAD HISTORY OF IIWWTP

The IIWWTP site has been preloaded in several phases prior to construction of the existing facilities. Major portion of the site has been preloaded prior to the original construction over a 2-year period from March 1959 to May 1961.

2.2.1 Initial Preload – 1959 to 1961

A technical paper by Ripley (1995), published in the Canadian Geotechnical Journal, describes the original preload at this site, settlement of the preload and post-constriction settlement from 1963 to 1987. The original preload plus surcharge height for the 1959-1961 phase varied from 0 to 8.7 m across the site. This height was understood to be with respect to the original site grade. Preload settlement, from February 1959 to April 1961, varied from 0.5 m to 1.7 m across the site. Post-construction settlement varied from 0.5 m to 0.7 m across the site over a period from April 1963 to June 1987. Table 3 summarizes the preload and settlement data from Ripley (1995).

Table 3. Summary of preload and settlement data*

Location	Preload + Surcharge Height above original site grade (m)	Preload Settlement (m)	Post-construction settlement From April 1963 to June 1987 (m)
Administration Building	8.5	1.7	0.5
Power Building	8.5	1.7	0.6
Digesters 1 & 2	4 to 6	1.1	0.7
Sedimentation Tanks 1 to 5	0.6 to 3.7	0.5 to 0.8	0.5 to 0.6
Pump Building	0 to 3.7	0.5	0.7
Maintenance Building	Not available	Not available	0.6 to 0.7
Access road	Not available	Not available	0.5 to 0.6

*Ref.: Ripley, C.F. (1995)

2.2.2 Subsequent Preloads for the Additions After 1961

It is understood that preloads with 2 to 6 months long duration were used for the construction of the various additions. No record of construction or settlement data of this later stage preloading is available to us.

2.2.3 Preloading for Future Additions

Preloading would have to be considered if there is a positive net loading from any future site filling or other developments at the IIWWTP site. If piling is to be considered, then the preload design should consider the increase in load distribution from the structures to the deeper compressible soil layers. Piles would transfer a significant portion of the loading from the structures to this deeper compressible layers. The height and duration of preload would depend on the net loading on foundation soils and area (in plan) of the proposed structure. For preliminary assessment it is recommended that a setback of 15m, from the nearest edge of an existing structure to the toe of the preload be considered. This could be re-evaluated on a case-by-case basis during the detailed design. Settlement plates would have to be installed (possibly with the preload contract) to monitor settlement of preload and possibly long-term settlement of the structures. The settlement plates would consists of surface settlement plates and deep plates at the top of soil units 2 and 3 (El. –0.5 and –8 m respectively).

Note that significant long-term settlement has occurred (and is expected to continue), since completion of the original construction in 1963. The measured settlement from April 1963 to June 1987 varied from 0.5 to 0.7 m, with approximately a maximum 75 mm of differential settlement over a distance of 30 m.

It is understood that repair works to rectify the distress caused by this settlement have been carried out on a regular basis. The observed distress include ground water leakage through failed expansion joints, cracks on the ground floor slab, basement walls and walkways between concrete sedimentation tanks. The repair work includes sealing the expansion joints, grouting below the ground floor slab etc.

For the design of any future additions, which would connect to the existing structures potential differential settlement between the existing and new structures would have to be considered. Careful planning and design of preload and foundations would be required.

2.3 BEARING CAPACITY OF EXISTING FOOTINGS

For evaluation of the existing footings bearing within the Unit 1 compact to dense river sand FILL, the following bearing pressure or resistance can be used:

- Allowable bearing pressure for working stress design - 125 kPa.
- Factored ultimate bearing resistance for seismic design - 190 kPa.

For the above recommendations it is assumed that the footings have a minimum width of 0.5 m and embedment of 0.6 m. Note that settlement would govern the design of larger footings. Geotechnical engineer should review the final loading conditions and footing design.

2.4 SEISMIC CONSIDERATIONS - IIWWTP

For seismic design, the recommendations given in the British Columbia Building Code (BCBC 1998) or National Building Code of Canada (NBCC 1995) could be used. The GVRD Engineering Standards Report on Seismic Design Criteria, dated Dec. 17, 2002, states that a wastewater treatment plant is expected to perform at “Life Safety Performance Level” under the NBCC 1:475 year design earthquake motion. The above noted GVRD guideline states that:

> *Water retaining structures in a ”Life Safety Performance Level” facility may experience distress, cracking and minor leakage but must remain operational at full capacity. Superstructures may experience significant but repairable damage but should not be in a condition of potential collapse. Repair of both the water retaining structures and the*

superstructure may require the facility to be out of service for up to 2 months during non-peak demand season.

According to the National Building Code of Canada (NBCC 1995, and British Columbia Building Code 1998), the IIWWTP site is located within seismic zones of Z_a=4 (acceleration) and Z_v=4 (velocity).

The recommended NBCC 475 year return period peak firm ground acceleration (PGA) and Peak Ground Velocity (PGV) for the site are 0.23g and 0.21 m/s respectively. The PGA and PGV given above are design horizontal motions on very dense till-like soils or soft rocks and g is the acceleration of gravity. The design earthquake, inferred from the building code, is a Magnitude 7.0 event.

It is understood that a new Building Code, which is due in 2005 (NBCC 2005), recommends Buildings be designed for 2475 year return period earthquake motion. The recommended PGA for the IIWWTP site would be 0.5g for the 2475 year return period (GSC Open File 4459 Table 1. Seismic hazard values intended for the 2005 NBCC "Design Data for Selected Locations in Canada" www.seismo.nrcan.gc.ca/hazards/OF4459/).

Based on discussions with GVSDD it is understood that:

- for the seismic assessment and upgrading of the existing structures recommendations given in the NBCC 1995 (475 year return period earthquake motion) is to be used;
- the NBCC 2005 (2475 year return period earthquake ground motions) is to be used for the design of any future structures.

2.4.1 Amplification of Ground Motion

The design firm ground motion could amplify when the earthquake induced shear waves propagate through the loose sands and silts of the Fraser River delta. To account for this ground motion amplification, a ground surface acceleration of 0.3g is recommended (Task Force Report, 1991) for use with the NBCC 1995 (475 year return period) design.

Ground motion amplification associated with the NBCC 2005 (2475 year return period) design earthquake is not available at this time. This would require detailed ground response analyses. The detailed analyses would also provide the required input parameters for liquefaction assessment and spectral accelerations for structural analyses.

2.4.2 Foundation Factor – NBCC 1995

A foundation factor (F) of 2.0 should be used in base shear calculations (Task Force Report, 1991) in conjunction with the NBCC 1995 design recommendations.

2.4.3 Liquefaction Assessment

When subjected to strong shaking, water saturated loose sands and non-plastic silts may lose their shear strength and behave like a heavy liquid, which is defined as liquefaction (National Research Council, 1985). Some of the consequences of liquefaction, related to this site are: post-liquefaction consolidation settlement; loss of foundation bearing capacity; settlement of heavy foundations; floatation of light underground tanks and; lateral spreading of the ground.

Significantly thick zones of potentially liquefiable loose sands exist within the IIWWTP site. Review of previous studies by Klohn Crippen, dated August 14, 1996 indicates that 2 to 4 m thick layers at 5 m to 11 m depth from the existing grade could liquefy due to the 1:475 year design earthquake motion. Thickness of the potentially liquefiable layer under the NBCC 2005 would not be available without the detailed ground response analyses described in section 2.3.1. However, for initial assessment the soil units 2 and 3 can be assumed liquefiable under the NBCC 2005 design earthquake motion (a total thickness in the order of 11 m).

Review of the previous studies, listed in Section 2.0, indicates that the calculated post-liquefaction ground settlement and lateral spread for the 1:475 year event is in the range of 200 mm and 300 mm respectively. This settlement would not be uniform across the site. For preliminary assessment the above noted settlement can be taken as differential over a horizontal distance of 5 m or between adjacent column footings.

For final design detailed drilling programs and analyses would have to be carried out to assess liquefaction potential and its consequences related specifically to each structure. This analysis would have to be carried out after completion of the ground response analyses described in section 2.3.1.

Note that the stability of the dykes and foreshore slopes may be inadequate, and therefore, some form of ground improvement along the waterfront may be required to prevent ground failure. Ground improvement for new structures could also be carried out to mitigate liquefaction. The improvement area should extent beyond the footprint of the structures. The extent and depth of improvement would have to be evaluated for each location/structure. Vibro-replacement sub-soil densification is commonly used to improve the ground in similar ground conditions.

2.4.4 Ground Improvement

Liquefaction of subsoil may cause instability and possible failure of the foreshore slopes around the IIWWTP as described above. This will likely cause distress and possible damage to the various structures at this site. Some form of ground improvement along the waterfront may be required to prevent ground and slope failure.

Our preliminary assessment indicates that a minimum 15 m wide zone around the perimeter of the existing IIWWTP facility would be effective in limiting the lateral spreading within tolerable limits. Note that liquefaction would still occur inside the non-densified area and below the existing structures. Therefore, post-liquefaction settlement, in the order of 200 mm would still be expected. Also, floatation of tanks may occur. In order to minimize settlement of buildings and the possible floatation of tanks other forms of remediation such as soil anchors/mini-piles can be considered.

We recommend that ground densification with vibro-replacement method be carried out to 14 m depth on a triangular grid pattern at 2.8 m center-to-center spacing.

A preliminary cost estimate for the 15 m wide densified berm would include the following:
Unit Cost = $8 per cubic metre of improved ground
Volume of improved ground: L x W x D = 1000 m x 15 m x 14 m = 210,000 cubic metre.
Estimated total cost of ground densification = $8 x 210,000 = $1,680,000.

The above cost estimate is based on the information obtained in 2004 from two local contractors and our recent experience of similar densification projects in the Fraser River delta.

For any new structures it is recommended that the footprint plus a 5 to 10 m wide envelope around the perimeter be densified. This is to mitigate liquefaction due to the design 475 year return period earthquake motion. A more detailed analysis would have to be carried out to determine the depth and extent of densification to mitigate liquefaction against the 2475 year design event. This analysis is beyond the current scope of work.

The above recommendations are preliminary and provided for planning purpose only. Detailed design and analysis would be required prior to any construction work and to provide an improved cost estimate.

2.4.5 Soil Anchors

Soil anchors can be considered for providing resistance against uplift of buildings and tanks. They can also be designed as mini-piles to provide additional axial compression capacity. The anchors can be installed within or around the perimeter of buildings, provided that enough headroom for the machinery is available – in the order of 3 m or more.

Use of soil anchors in areas where the ground water table is closer to the surface would require careful design considerations. Protection of anchors against corrosion and provision of waterproofing at the base slab would be critical. Also, long-term settlement of the structures relative to the anchors would have to be evaluated for the design of anchor head.

If the anchors are designed to carry compression loading, then a significant portion of the load from the structures would be transferred to the deeper compressible silty soils. This would cause additional long-term settlement and would have to be considered during the design of preload for new structures. Similarly, design of piles to provide compression capacity for the existing structures would have to consider this potential settlement problem.

Typically, these anchors would consist of Double Corrosion Protected (DCP) Dywidag bars of Grade 517 (75 ksi) or better. The bars would be grouted in approximately 125 mm diameter drilled holes.

Length of the bars would typically be in the order of 20 m, depending on the ground condition and the required capacity. Also, the anchors would have to be installed with steel casing to prevent buckling and/or shear failure under lateral movement of the ground.

A factored ultimate anchor bond capacity of 22 kN/m (1.5 kips/foot) can be used for design for anchors installed with after-grout hardware. Liquefaction of subsoils would lead to reduction in anchor capacity. It is recommended that the thickness of potentially liquefiable layer (maximum 4 m thickness for 475 year return period, preliminary) be subtracted from the bond length in the anchor capacity calculations. Liquefiable layer thickness, anchors bond length etc. would have to evaluated during detailed design.

All anchors would have to be proof tested after installation to confirm the design capacity. Geotechnical engineer should review the design. The above noted capacity is for tensile loading only. However, the anchors would also improve the compression capacity of the foundation. If required, anchors can be designed to take compression loading (mini-piles).

A preliminary cost estimate for soil anchors would be $200 per metre length (or $200/m x 20 m = $4000 per anchor) for supply and installation.

An alternative to soil anchors would be 300 to 500 mm diameter steel or concrete piles. If piles are chosen, our recommendation is to use concrete filled steel pipe piles as they are expected to have more ductility and perform better under seismic loading conditions than a concrete pile of equivalent diameter. Driving piles next to the existing buildings would have to be assessed carefully as some buildings may be vibration sensitive. Pre-drilling could be considered to reduce vibration. For preliminary design, an allowable axial capacity in the order of 300 and 500 kN per pile can be considered for the 300 mm and 500 mm diameter piles with 15 m embedment. Piles designed with the above recommendation should perform satisfactorily under seismic conditions with 475 year return period earthquake motion. However, the above assumption would have to be verified during the design. Similar to the anchors/minipiles, any compression loading on the piles would transfer a significant portion of the loading from the structure to the deep compressible soil layers. This loading would have to be considered in the preload design.

2.4.6 Lateral Earth Pressure

Lateral earth pressure diagram shown in Dwg. 2 is recommended for assessment of the existing structures. For the calculations the following assumptions were made:

- Basement and below grade tank walls are rigid;
- Level ground condition and free draining backfill;
- Backfill unit weight is 19 kN/m^3;
- At rest earth pressure coefficient K_o = 0.5;
- Seismic coefficient k_h = 0.3 for use in Wood's solution for non-yielding rigid walls.

Note that the pressures given in Dwg. 2 are unfactored and assume pre-liquefaction soil conditions. Also, note that the seismic pressure distribution recommended by Wood (1973) is a parabolic curve. The recommendation shown in Dwg. 2 is a simplified pressure distribution with an inverted triangular distribution.

2.5 UPLIFT PRESSURE ON UNDERGROUND FLOOR SLAB

For calculation of uplift pressure on underground floor slabs a groundwater table elevation of +2m can be used. The above recommendation is based on the highest measured groundwater table level (+1.2m) at the site plus an increase of 450 mm for potential rise in sea level over the next 50 years (see section 4.0) plus another 350 mm to account for uncertainties in measured water table levels.

Emptying the deeper tanks, such as the digesters for maintenance work would have to be carefully planned to avoid high ground water gradients immediately below the base of the tanks.

Liquefaction of soils below the deeper tanks may cause additional uplift pressures on the base of the tanks. The uplift pressure would be due to the liquefied soil behaving like a heavy liquid (with unit weight in the order of 18 kN/m^3). Buoyant weight of the tank and its contents, and shear resistance from the non-liquefied soils along the perimeter of the tank would provide resistance against uplift. This would have to be assessed using a detailed liquefaction assessment with test hole investigation, preferably using Cone Penetration Test holes. The test holes can be drilled outside the existing tanks and/or within the footprint of any new structures. If the assessment indicates potential instability due to uplift pressures, then increasing the dead load of the tank using heavy slabs could be considered. Addition of any new load would have to be evaluated against potential long-term settlement due to consolidation of the deep compressible soil layers. Soil anchors could also be considered as discussed in section 2.4.5 to provide resistance against uplift pressures.

3 LIONS GATE WASTEWATER TREATMENT PLANT (LLWWTP)

The Lions Gate Wastewater Treatment Plant (LGWWTP) was opened in 1961 and has been expanded several times over the years. The discussions given in the following sections are based on a review of the following documents:

- GVRD Memorandum, Lions Gate WWTP-Disinfection System Upgrade, Geotechnical Recommendations, dated August 08, 2002;
- Terra Engineering Ltd report, Headwprks Upgrade Project, Lions Gate Wastewater Treatment Plant, Geotechnical Study, dated November, 1998;
- Technical paper by E. Naesgaard and M.Uthayakumar (1999). Numerical Analyses for Seismic Retrofit Design, Lions Gate Bridge, Vancouver, British Columbia. Proceedings of the International *FLAC* Symposium on Numerical Modeling in Geomechnics, Minneapolis, Minnesota, pp.349-356.

3.1 SITE AND SUBSOIL CONDITIONS - LGWWTP

The LGWWTP is located at 100 Bridge Road, North Vancouver, B.C. and bounded by the Lions Gate Bridge North Approach on the east, by the B.C. Rail Right of Way on the north and by Burrard Inlet on the south and south-west. The Plant includes the following buildings, tanks and other facilities:

- 4 circular digesters;
- 3 sludge control buildings;
- a sludge thickener;
- a sludge dewatering building;
- a filter screen building;
- a pump building;
- 8 sedimentation tanks;
- an operations building;
- an “Engineering Building”;
- maintenance and equipment shop buildings.

3.1.1 Site Development

It is understood that the LGWWTP site was developed in several stages, starting from 1959. The original site preparation was carried out (in 1959) by clearing and grubbing of all trees, brush, rubbish, stumps, roots and logs. The area was then stripped of all organic topsoil, silt, roots and logs to expose the underlying sand and gravel deposits. Granular fill was then placed and compacted to bring the grade to the current elevation.

It is understood that the original ground elevation, prior to any development varied from El. 2.63 m at the north end of the site to about El. 1.72 m towards the south end. Current grade elevation varies from approximately 2.6 m to 4 m across the site. All elevations noted above are geodetic.

3.1.2 Subsoil Conditions

Subsoil conditions and profile were obtained from the various test holes drilled in the past at this site. Also, test hole information from our Geotechnical Report for the upgrading of the Lions Gate Bridge North Approach Piers was utilized.

Generalized sub-soil profile with increasing depth from these test holes is given in Table 4.

Table 4. Generalized sub-soil profile, LLWWTP

Soil units	Approximate Thickness, (m)	Description
Unit 1	0 to 1.8	FILL - sand and gravel with pieces of wood, debris, and organics
Unit 2	13 to 15	SAND and GRAVEL with some cobbles and boulders to a depth of approximately 15 m
Unit 3	25 to 35	silty SAND with some gravel to 40 to 50 m depth
Unit 4	20 to 40	sandy SILT to sandy CLAY with gravel and cobbles (glacial till) to depth of 70 m to 80 m depth over
Unit 5	> 150	CLAYSTONE bedrock to depths over 220 m.

3.1.3 Groundwater Level

Groundwater level measured using piezometers at various test holes indicate a range of El. –0.2 m to +0.9 m. Piezometer readings and visual observation during the various phases of investigation and construction indicate that the groundwater at this site varied with the tide level in the Burrard Inlet, amount of precipitation and also experiences seasonal effects, with higher levels in wet Winter months and lower levels during the drier Summer and early Fall months.

Note that the groundwater and sub-soil profile conditions described above reflect conditions at the test hole locations on the day of measurements only and may not be representative across the site.

3.2 SEISMIC CONSIDERATIONS - LGWWTP

According to NBCC 1995 (and BCBC 1998), the LGWWTP site is located within seismic zones of Z_a=4 (acceleration) and Z_v=4 (velocity).

The recommended NBCC 475 year return period Peak firm Ground Acceleration (PGA) and Peak Ground Velocity (PGV) for the site are 0.23g and 0.21 m/s respectively.

The PGA and PGV given above are design horizontal motions on very dense till-like soils or soft rocks and g is the acceleration of gravity. The design earthquake, inferred from the building code, is a Magnitude 7.0 event.

PGA for the LGWWTP site would be 0.44g for the 2475 year return period earthquake motion (NBCC2005, GSC Open File 4459 Table 1. Seismic hazard values intended for the 2005 NBCC "Design Data for Selected Locations in Canada" www.seismo.nrcan.gc.ca/hazards/OF4459/).

3.2.1 Amplification of Ground Motion

Experience from ground response analyses carried out for the upgrading of the Lions Gate Bridge North Approach indicate that some marginal amplification of ground motion may occur when the earthquake induced shear waves propagate through sand, silt and clayey soils. Surface PGA of 0.25g is recommended for design (475 year return period motion).

Ground motion amplification associated with the NBCC 2005 (2475 year return period) design earthquake is not available at this time. This would require detailed ground response analyses. The detailed analyses would also provide the required input parameters for liquefaction assessment and spectral accelerations for structural analyses.

3.2.2 Foundation Factor

To account for the above noted ground motion amplification, a foundation factor (F) of 1.3 should be used in base shear calculations in conjunction with the NBCC 1995 (1:475 year return period) design recommendations.

3.2.3 Liquefaction Assessment

For the design 475 year return period earthquake motion, potential liquefiable zones are expected to be scattered sporadically throughout the site, with some local zones of significant liquefaction.

Review of previous studies, listed in Section 3, indicates that the calculated post-liquefaction ground settlement is in the range of 0 to 250 mm. This settlement would not be uniform across the site. For preliminary assessment half of the above noted settlement (125 mm) can be taken as differential over a horizontal distance of 5 m or between adjacent column footings.

Earthquake shaking together with subsoil liquefaction will cause the ground to move towards the Burrard Inlet. This movement, referred as lateral spreading, is highest near the water edge and decreases with increasing distance from the water edge. Previous studies indicate lateral spread of approximately 0.3 m at a distance of 50 m to 100 m from the water-edge. The magnitude of the calculated lateral spread approaches zero at the north-east end of the site.

Detailed drilling program and analyses would have to be carried out to assess liquefaction potential and its consequences related to each structure for the final design.

3.2.4 Ground Improvement

The above noted lateral spreading of the ground towards Burrard Inlet may cause distress within the existing structures. Some form of ground improvement within limited zone along the waterfront could prevent lateral spreading of the ground towards the inlet.

For preliminary design it is recommended that a 15 m wide densified berm along the length of the south and south-west boundary of the site be considered. Densification would have to be carried out from the existing grade to a depth of 15 m. Densification using vibro-flotation or replacement can be considered. Presence of possible boulders within the Unit 2 soil layer may prevent penetration of the vibrating probe. To assess this problem a densification test section, 15 m x 15 m in plan area, is recommended.

“Dynamic compaction” can also be considered for ground densification. This method involves repeated dropping of a 10 to 20 tonne weight from heights of 15 to 25 m. However, vibration from this method of densification can be damaging to the existing structures and to the adjacent Lions Gate Bridge pier foundations.

The purpose of the densified berm is to reduce the amount of liquefaction induced lateral movement of the ground towards Burrard Inlet. Note that liquefaction would still occur inside the non-densified area of the site and below the existing structures. Therefore, post-liquefaction settlement, in the order of 0 to 250 mm would still be expected. Also, floatation of buried tanks may occur. To prevent settlement of buildings and floatation of tanks other forms of remediation such as soil anchors/mini-piles can be considered.

A ground densification program using stone columns (vibro-replacement) could be considered. For preliminary considerations stone columns using a triangular grid pattern at 2.8 m center-to-center spacing can be used. Note that this is a preliminary recommendation for cost estimating purpose only and assumes that the boulders in the sub-soils do not prevent the penetration of vibro-replacement probe to the required depth.

Approximate cost for the 15 m wide densified berm is estimated as follows:
Unit Cost = $12 per cubic metre of improved ground. The higher unit cost (than that for IIWWTP) is to account for the presence of gravels, cobbles and boulders at this site. This unit cost is based on the price quoted for densification of a site with similar soil conditions.
Volume of improved ground: L x W x D = 400 m x 15 m x 15 m = 90,000 cubic metre.
Cost estimate = $12 x 90,000 = $1,080,000.

For any new structures within the site it is recommended that the footprint plus 5 to 10 m wide envelope around the perimeter be densified.

The above recommendations are preliminary and provided for planning purpose only. Detailed design and analysis would be required prior to any construction work.

3.2.5 Soil Anchors

Soil anchors can be considered for providing resistance against uplift of buildings and tanks. They can also be designed as mini-piles to provide additional axial compression capacity. These anchors can be installed within or around the perimeter of the building, provided that enough headroom for the machinery is available – in the order of 3 m or more. Protection of anchors against corrosion and provision of waterproofing at the base slab would be critical. Also, long-term settlement of the structures relative to the anchors would have to be evaluated for the design of anchor head.

Typically, these anchors would consist of Double Corrosion Protected (DCP) Dywidag bars of Grade 517 (75 ksi) or better. The bars would be grouted in approximately 125 mm diameter drilled holes. Grouted length of the bars would typically be in the order of 20 m, depending on the ground condition and the required capacity.

For design, a factored ultimate anchor bond capacity of 35 kN/m (2.4 kips/foot) can be used for anchors installed with after-grout hardware. Liquefaction of subsoils would lead to reduction in anchor capacity. It is recommended that the thickness of potentially liquefiable layer (maximum 3 m thickness for 475 year return period earthquake motion – preliminary) be subtracted from the bond length in the anchor capacity calculations. Liquefiable layer thickness, anchors bond length etc. would have to evaluated during detailed design.

All anchors would have to be proof tested after installation to confirm the design capacity. Geotechnical engineer should review the design. The above noted capacity is for tensile loading only. However, the anchors would also improve the compression capacity of the foundation. If required, anchors can be designed to take compression loading (mini-piles).

3.2.6 Lateral Earth Pressure

Lateral earth pressure diagram shown in Dwg. 3 is recommended for the assessment of the existing structures. For the calculations the following assumptions were made:

- Basement and below grade tank walls are rigid;
- Level ground condition and free draining backfill;
- Backfill unit weight is 19 kN/m^3;
- At rest earth pressure coefficient K_o = 0.5;
- Seismic coefficients k_h = 0.25 for use in Wood's solution for non-yielding rigid walls.

Note that the pressures given in Dwg. 3 are unfactored and assume pre-liquefaction soil conditions. The seismic pressure distribution recommended by Wood (1973) is a parabolic curve. This is simplified using an inverted triangle as shown in Dwg. 3.

3.3 UPLIFT PRESSURE ON UNDERGROUND FLOOR SLAB

For calculation of uplift pressure on underground floor slabs a groundwater table elevation of +2m can be used. Empting the deeper tanks, such as the digesters for maintenance work would have to be carefully planned to avoid high ground water gradients immediately below the base of the tanks.

4 EFFECTS OF RISE IN SEA LEVEL

It is understood that the effects of potential rise in sea level are to be considered in this assessment. Recommended guidelines by GVRD states that 2 to 9 mm rise per year could be considered (This would result in a sea level rise of about 450 mm over a 50 year period). Note that the existing site grade elevation is approximately 4.0 m, geodetic at IIWWTP and 2.6 m to 4 m at LGWWTP.

A review of the Sea Island Dyke Upgrading project reports (Kerr Wood Leidal report dated October 1996 for YVR) indicates that the design dyke crest elevation around the Sea Island is 3.5 m, geodetic. It is understood that the above crest elevation provides for the 200-year return period still water level, plus 0.6 m freeboard. An additional 0.3 m of freeboard (to a crest elevation of 3.8 m) was recommended to account for the “possible rise in mean sea level from long term climatic changes (global warming)”.

Groundwater level at both LGWWTP and IIWWTP sites are closely related to the tidal variation. Calculation of basement wall pressures of the various structures would have to consider the potential increase in water table level. The lateral earth pressure diagrams shown in Dwgs. 2 and 3 considers a 450 mm rise in ground water level above the highest measured groundwater level at the two sites.

5 PRELIMINARY PIPELINE ROUTE ASSESSMENT

Trow has conducted a preliminary pipeline route assessment of proposed routes from LGWWTP to IIWWTP. The purpose of the assessment is to develop a tool to guide the proposed development of new pipeline routes for tie-in to the existing/upgraded sewer system.

5.1 SCOPE OF WORK

The scope of work includes a review of available information, geology, assessment of routings proposed by others, and construction feasibility.

Information provided includes:

- navigational charts and project map area by Dayton & Knight Ltd and;
- mapping of the *"Subtidal Biophysical Inventory Maps of Burrard Inlet…"* by FORESHORE TECHNOLOGIES INCORPORATED (FTI).

5.2 PROPOSED PIPELINE ROUTING

The project includes several proposed pipeline routing options for installation of a 1.2 m outside diameter steel pipe to carry sewage from LGWWTP to IIWWTP through Vancouver. The proposed pipelines vary in length depending on where in Vancouver (or Iona Island) they tie-in to the existing/upgraded network.

From the LGWWTP, four landfall points have been pre-selected by others, as possible connecting locations to the existing network. Each landfall point is near an existing or upgradeable sewage system. Drawing 1 shows the proposed routes and the landfall site locations. With the exception of Route 3, which has a terrestrial component, all others are predominantly marine routes. Also, Route 1 is the only route proposed with a landfall along the southern coast of Vancouver. All other routing options have landfalls that are located along the north shore of Vancouver. The landfall site locations and routing options are described in the following sub-sections. The pipeline routes have been assessed in terms of the general soil conditions, environmental impact, and topography.

5.2.1 Route 1

Route 1 is a marine route, passing near the vicinity of the Lions Gate Bridge through Burrard Inlet and around Point Grey up to the Iona Island Outfall. The Outfall is located on the north shore of Iona Island. The total distance of this route is approximately 12.5km.

The pipeline alignment faces some challenging topography under water through First Narrows and Outer Harbour. The First Narrows crossing and pipeline placement on the East end of the Outer Harbour will likely require working at submarine depths of 22 to 53 meters.

The sub-bottom of Outer Harbour is assumed to be similar to the surficial mud found along the northwestern area of Vancouver. Trenching of the sub bottom material may be relatively easy, but turbidity from the trenching may be problematic. And, slope stability may be an issue specially when considering the high current speed reported for the area near First Narrows.

The area immediately west of Point Grey is reported as mostly sand. The soil information along the shores of the North Arm of the Fraser River indicates the probability of very loose to medium dense river sand, compressible silts and clays over very dense Pleistocene sediments or soft sedimentary rock. Little difficulty with pipe trenching is expected in this area. However, this is an environmentally sensitive and recreational area. More specific details are needed before considering pipe placement in this region.

The North Arm of the Fraser River supports primarily coastal navigation. Thus, pipe placement may have to infringe on the mud flats that bound the channel should the width of the channel not support construction activities and continuance of navigation.

5.2.2 Route 2

Route 2 is a marine route from the vicinity of the Lions Gate Bridge through Burrard Inlet with landfall on the southern shore of English Bay near Balaclava Street. The total distance of this route is approximately 4.2km. The proposed Route 2 would tie-in with the existing Highburry Interceptor on a landfall on the southern shores of English Bay. The Highburry Interceptor runs north to south from English Bay to Iona Island Outfall (see Dwg. 1).

Review of general geology shows that most of the offshore area northwest of Stanley Park is made up of sand with localized gravel, cobbles and boulders. Occasionally, bedrock extends to the surface on the eastern sections of the Outer Bay (English Bay). The main constituent in the surficial material at English Bay is silts, clays and sea bottom mud. Some glacial marine sediments and till may be present over a soft sedimentary bedrock. Trenching may be relatively easy, but turbidity from the trenching may be problematic. The landfall area may experience variable overburden depth comprised of sands over till or sedimentary bedrock. The bathymetry of the English Bay appears suitable to pipe placement, relatively constant shallow water depth. However, the initial drop off associated with the First Narrows still requires caution. As mentioned earlier, the sub-bottom in this area is very steep and transitions to depth may approach 50-meters. The presence of high water currents will also affect the stability and pipe placement.

5.2.3 Route 3

Route 3 is a primarily terrestrial route that crosses the First Narrows near the Lions Gate Bridge and makes a landfall in Stanley Park near Prospect Point. The route then follows Highway 99 through Stanley Park into downtown Vancouver, proceeds along Denman Street and runs southwest until it reaches Beach Avenue and the Seawall Promenade, then follows Beach Avenue south until it reaches the tie-in at West End Interceptor No. 2 in Downtown Vancouver. The total distance of this route is approximately 2.7km. Approximately 440 meters of this route is a marine lay, which constitutes nearly 15 percent of the total length.

The tie-in with the West End Interceptor connects the two plants, (LGWWTP and IIWWTP) through the Eight Avenue Interceptor and Highburry Interceptor.

In the vicinity of Stanley Park, the overburden generally consist of sands over till or sedimentary bedrock at relatively shallow depth.

Collocation with Highway 99, would allow access to the north end of downtown Vancouver. The overall impacts to Stanley Park associated with construction could be reduced with the collocation. A Construction corridor through downtown Vancouver appears to be feasible from constructability. Microtunneling may be a viable construction method to navigate the 1.2-m steel pipe through this densely developed area. The overburden in downtown Vancouver is primarily glacial marine soils, till-like, to clayey silts over soft sedimentary bedrock. Microtunneling requires intermediate jacking stations. The construction of the jacking stations will result in localized disruption to downtown Vancouver. The estimated linear impact is approximately 2500 meters. The spacing of the jacking stations can be estimated at 200 meters. This distance suggests that a several jacking stations may be needed to traverse downtown Vancouver.

Jack & bore pipe installation methods can be considered in the immediate vicinity of downtown Vancouver. The use of microtunnel technique, which will allow for longer distances between pits than Jack & bore allows, should also be considered. The impacts of construction along congested city streets would have to be evaluated. Underground utilities may also prove challenging to any installation technique in this area.

The overburden on the southern shores of North Vancouver is comprised of medium dense sand and gravel with cobbles with occasional loose sandy soil zones. The cobbles with localized boulder zones extend nearly 90 percent across the Narrows to the south of North Vancouver, west of the Lions Gate Bridge. East of the bridge, the surficial cobble zone spans the entire width of the crossing.

The reported subsurface conditions may be challenging, or even detrimental, for the use of Horizontal Directional Drilling (HDD) technology. The reported depths in the Narrows exceed 15 meters below the water surface at low tide. Presence of cobbles and boulders poses difficulty for any HDD installation. Jack & bore and microtunnel likewise may have trouble under similar conditions.

The south side of the Narrows at Prospect Point has strata similar to the north side, but with sands dominating the area immediately to the west of the Lions Gate Bridge and bedrock immediately to the east. It is suggested that a landfall, if considering an HDD, be moved west of the Lions Gate Bridge to provide an exit through the reported sandy subsurface.

5.2.4 Route 4

Route 4 is a marine route across the Inner Harbour. It crosses the navigable inlet east of the Lions Gate Bridge and Stanley Park. This pipeline alignment makes landfall near the Harbour West Interceptor in the general vicinity of Columbia Street. The total distance of this route is approximately 4.0km. From the Harbour West Interceptor, the routing continues via Clarke Drive Interceptor, Eight Avenue Interceptor and Highbury Interceptor to the IIWWTP.

The depth of the Inner Harbour can exceed 62 meters in the vicinity of the proposed pipeline alignment. On the southern shores of the Inner Harbour, some localized boulders dominate the sub-bottom.

Table 5 shows a preliminary route ranking for the proposed routes (Rank 1 for the best route). The evaluation criteria included total route length, offshore construction costs, collocation, and environmental impacts.

Table 5. Preliminary Route Ranking

Ranking	Route No.	Route Length (km)	Advantages	Disadvantages
4	1	12.5	-no new onshore pipeline	-longest routing option -offshore construction -land to water HDD (at least one)
2	2	4.2	-no new onshore pipeline	-land to water HDD (2) -offshore construction
3	3	2.8	-land to Land HDD -no offshore construction	-requires construction of new onshore pipeline (approximately 2.3-Km) -new onshore pipeline would affect parks and populated areas -construction through downtown Vancouver
1	4	3.2	-no new onshore pipeline -shortest offshore route distance	-land to water HDD (2) -offshore construction

5.3 PIPELINE CONSTRUCTION - ONSHORE CONSTRUCTION TECHNIQUES

The following sections provide a summary of conventional methodology for pipe placement as well as a brief summary and details of suggested HDDs and microtunneling alternatives. Table 6 summarized the construction techniques and highlights some of the advantage/disadvantages for each.

5.3.1 Conventional Lay

The preferred construction method for the installation of pipeline is open trench or open cut. However, this method is not always suitable since it requires extensive working room. Construction width can be as great as 30 – 35 m. However, short runs can be accomplished in significantly reduced widths ranging from 15 – 18 m.

5.3.2 Horizontal Directional Drill (HDD)

Use of HDDs to make landfall and to cross First Narrows is probably the best way to minimize impacts to the region's natural sensitive habitats, and commercial and recreational activities. A summary of the suggested HDDs follows:

- HDD No. 1: From LGWWTP to offshore;
- HDD No. 2: From offshore to Iona Island;
- HDD No. 3: From English Bay to landfall near English Bay Interceptor;
- HDD No. 4: From LGWWTP to Stanley Park;
- HDD No. 5: From LGWWTP to bottom of Inner Harbour;
- HDD No. 6: From bottom of Inner Harbour to landfall near Harbor West Interceptor.

With the exception of HDD No. 5, the proposed HDD exit points are located in a marine environment. Typically, this construction is acceptable when sufficient pullback is available. The entry point will be onshore and the exit point will be offshore, or underwater. The pullback section would be assembled and floated offshore. This section will then be tied-in and pullback through the drilled path.

HDDs techniques to gap distances in the coastal regions may experience boulders at depth. If boulders are encountered, this technique will have trouble and could be fatal if boulders cannot be removed or retrieved from the drilled path. Soft cohesive soils, loose cohesionless soils, and high ground water table also require careful consideration. However, HDD construction is still possible with proper geotechnical investigation and design.

5.3.3 Microtunneling

Microtunneling is an option that could be used for construction through heavily developed areas. It is a viable alternative to HDD with limited pullback area and to lengthy Jack & Bore in restricted space.

However, this installation is relatively costly and therefore should be considered only in areas where HDD is not suitable.

5.4 PIPELINE CONSTRUCTION - OFFSHORE/MARINE CONSTRUCTION

From the LGWWTP to all the proposed tie-in locations, the pipeline would traverse a marine environment. In addition to trenchless technologies, there are two basic options for marine burial: excavating a trench or; laying the pipeline on the seafloor and covering with concrete mats or loose rocks. Both options are discussed below.

5.4.1 Trenching

From the LGWWTP HDD exit hole, to the landfall HDD entry, the pipeline could be installed in a trench in the seafloor. The trench could be excavated using jetting, where possible. In hard bottom areas, or in areas where jetting is not possible, the trench would have to be excavated. After the pipeline has been installed, rock would be placed over the pipeline in the trench for cover. The interstitial spaces between the rocks are expected to fill as the currents transport material across the trench line after construction. The excavated material would be allowed to dissipate with the local currents. Trenching may provide pipeline stability in the currents off Vancouver - This needs further evaluation and design consideration. Trenching is expected have impacts to the seafloor. Potentially, some impacts to biota would be offset by the re-colonization of the same on the pipeline trench after construction

5.4.2 Lay Pipeline on Seafloor and Cover

The pipeline can be laid on the seafloor and covered with concrete mats or rock. This would result in an approximately 12 m wide coverage of rock, with the apex over the pipeline approximately 1.8 m high. This option is not recommended for the following reasons:

- Increased construction costs and;
- Concern that boat anchors or other types of seafloor activities would dislodge the rock and eventually the pipeline.

Table 6. Summary of Construction Techniques.

Methods	Advantages	Disadvantages
Onshore Conventional Lay/Open-Cut	- Prior to backfilling, easy access to pipe product for inspection. - Cost-effective if there are no obstacles such as waterbodies, roads, etc..	- Larger crews needed . - Traffic management may be required. - Often impacts environment, or excavation may be prohibited in protected wetlands. - Permits needed for cutting through roads, wetlands, and traffic management. - Open trench creates hazards. - Within confined areas, the excavations may require support. - Dewatering may be required in areas with a high water table.
Trenching Offshore	- Pipe is buried, so that it may be stable/protected from currents. - Provides a greater degree of safety from ship anchors.	- Larger impact to the seafloor. - Increase in turbidity may result in sedimentation down current of the construction zone. - Impacts to established inter- and immediate subtidal habitat.
Jack & Bore	-Reduced impact, if properly constructed.	-Limited boring/jacking distances. -Access pits may require earth support and/or dewatering.
Micro-tunneling	- Reduced surficial working area footprint. - Minimum surface equipment required. - Minimum equipment noise and emissions. - Minimum traffic disturbance. - Minimum restoration in access pit areas.	- Cost. - Longer distances may require intermediate jacking stations. - Drilling head and equipment may be hard to repair or recover if problems or obstacles are encountered.
Horizontal Directional Drill, HDD	- No direct interference with roads, waterbodies, monuments, landscapes, driveways, or wetlands - Minimal impact on environmentally-sensitive areas. - Minimal need for excavations, except at entry and exit points if required. - Faster than conventional methods or other trenchless technologies (Microtunnel or Jack & bore). - Reduces restoration cost and environmental impact. - Minimal traffic disruption. - Reduces the need to expose other utilities. - Eases the process of obtaining road or wetland crossing permits.	- Cost. - Not applicable to all subsurface conditions. - Requires workspace for pullback area. - Potential for inadvertent mud flow returns (frac-outs).

5.5 DISCUSSION

5.5.1 General

The geology and the geotechnical properties of the soils in the area need to be explored further. Detailed assessment of liquefaction and submarine mudslides potentials would have to be assessed. Variation in groundwater table level would have to be considered for the construction of pits or ditches.

The presence of boulders and cobbles as the main substrate will make any type of construction difficult. Trenchless methods, like HDD, may be impossible to implement under these conditions, unless the uniformity of the material is known in more detail.

The depth of marine lay across the Inner Harbour and Outer Harbour can exceed 62 meters.

Slope stability needs to be addressed in the marine environments and the route selection should consider the stability of the marine slopes, especially where the route parallels a slope instead of ascending or descending the same.

The property difference between the transitions of cobble/boulder to loose, water-saturated materials like sand/mud may induce stresses on the pipeline due to differential settlement. Detailed design should consider long term and differential settlement along the pipeline alignment.

5.5.2 Construction Limitations

The construction of a sewer pipeline in the project area has several unique geotechnical challenges. The following would have to be obtained or addressed during the detailed design:

- Depth of water;
- Differential settlement;
- Dewatering of open pits and/or trenches;
- Current velocities;
- Wave heights;
- Erosion due to wave action;
- Slope stability;
- Marine cover and;
- Environmental impacts.

Additional concerns may arise during the implementation of the project that may require site-specific design and/or further investigation. For instance, navigational concerns would likely preclude a crossing directly in the Narrows. Consideration would have to be made of shoreline crossing either west of the Capilano River or east of Vancouver Wharves. Another concern is that trenching is presently considered ocean disposal by Environment Canada (although there are some hints of policy change from the Atlantic Region) and some of the sediments will likely exceed ocean disposal standards for cadmium and polycyclic aromatic hydrocarbons.

6 CONCLUSIONS

Results of the preliminary geotechnical assessment and design recommendations for the upgrading of the Iona Island and Lions Gate Wastewater Treatment Plants are given in this report. The purpose of this preliminary assessment is to prepare facility plans for the above noted plants in accordance with the approved Liquid Waste Management Plan of the Greater Vancouver Sewerage & Drainage District.

The assessment results given in this report are provided for planning purpose only. Detailed design and analysis would be needed for the final design. The detailed analysis would require subsoil data, which would have to be obtained from site specific drilling methods. The analysis would include liquefaction assessment, estimation of seismically induced ground deformation, foundation bearing capacity, settlement etc.

We hope that the information given in this report is sufficient for your current needs. If you have any questions please do not hesitate to call the undersigned.

Trow Associates Inc.

M. Uthayakumar

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Dec. 20, 2004

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REFERENCE

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Interpretation and Use of Study and Report

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering consulting practices in this area. No other warranty, expressed or implied, is made. Engineering studies and reports do not include environmental consulting unless specifically stated in the engineering report.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF THE REPORT

The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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The information and opinions expressed in the Report, or any document forming the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS “APPROVED USERS”. The contents of the Report remain our copyright property and we authorise only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorised use of the Report.

5. INTERPRETATION OF THE REPORT

a. Nature and Exactness of Descriptions: Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations, or building envelope descriptions, utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarising such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b. Reliance on Provided information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.

c. To avoid misunderstandings, Trow should be retained to work with the other design professionals to explain relevant engineering findings and to review their plans, drawings, and specifications relative to engineering issues pertaining to consulting services provided by Trow. Further, Trow should be retained to provide field reviews during the construction, consistent with building codes guidelines and generally accepted practices. Where applicable, the field services recommended for the project are the minimum necessary to ascertain that the Contractor’s work is being carried out in general conformity with Trow’s recommendations. Any reduction from the level of services normally recommended will result in Trow providing qualified opinions regarding adequacy of the work.

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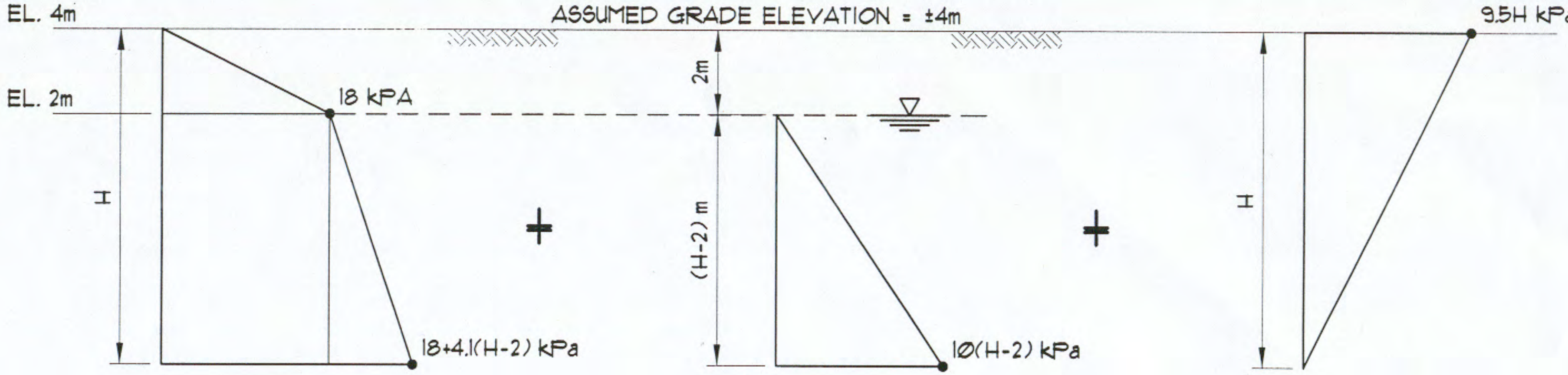

STATIC EARTH PRESSURE

HYDROSTATIC PRESSURE

SEISMIC PRESSURE
(SIMPLIFIED)

NOTES:
(1) ALL PRESSURES ARE IN kPa, AND DEPTH ARE IN METRES.
(2) PRESSURES GIVEN ABOVE ARE UNFACTORED.
(3) ALL THREE LOADS (STATIC, HYDROSTATIC AND SEISMIC) ARE ADDITIVE WITH APPROPRIATE LOAD FACTORS.
(4) THIS PRESSURE DIAGRAM SHOULD NOT BE USED FOR NEW WALL DESIGN.
(5) COMPACTION INDUCED AND SURCHARGE PRESSURE DURING CONSTRUCTION WOULD BE ADDITIONAL FOR NEW WALLS.

ASSUMPTIONS:
(1) WALL IS RIGID: WALL HEIGHT $_3$= Hm BELOW ADJACENT EXISTING GRADE.
(2) WATER TABLE DEPTH = 2m BELOW ADJACENT EXISTING GRADE.
(3) UNIT WEIGHT OF SOIL = 19kN/m , AT-REST EARTH PRESSURE COEFFICIENT K_o=0.5
(4) SEISMIC COEFFICIENT = 0.25

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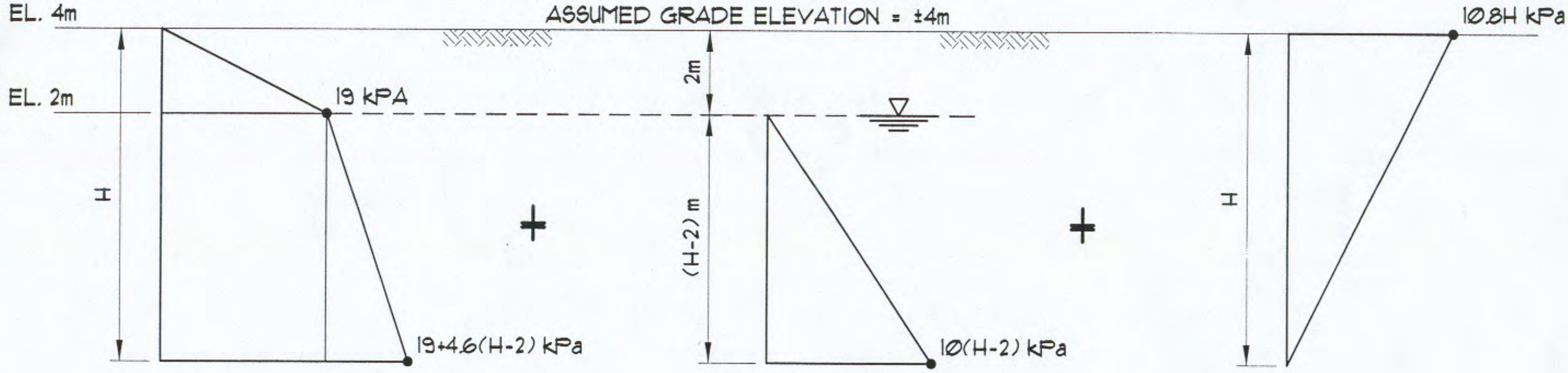
PROJECT NO. APR. 28/04	DFTR. MG	DSGN.	CHK. MU	
TITLE: LATERAL EARTH PRESSURE DIAGRAM LIONS GATE WASTEWATER TREATMENT PLANT				
DATE APR. 28/04	SCALE: NTS	DWG NO. 2		



STATIC EARTH PRESSURE

HYDROSTATIC PRESSURE

SEISMIC PRESSURE
(SIMPLIFIED)

NOTES:
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ASSUMPTIONS:
(1) WALL IS RIGID: WALL HEIGHT $_3$= Hm BELOW ADJACENT EXISTING GRADE.
(2) WATER TABLE DEPTH = 2m BELOW ADJACENT EXISTING GRADE.
(3) UNIT WEIGHT OF SOIL = 18kN/m , AT-REST EARTH PRESSURE COEFFICIENT K_o=0.5
(4) SEISMIC COEFFICIENT = 0.3

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PROJECT: FACILITY PLANS FOR IONA ISLAND & LIONS GATE WASTEWATER TREATMENT PLANTS

TITLE: LATERAL EARTH PRESSURE DIAGRAM IONA ISLAND WASTEWATER TREATMENT PLANT

PROJECT NO.	DFTR.	DSGN.	CHK.	DATE	SCALE:	DWG NO.
021-05499	MG		MU	APR. 28/04	NTS	3

GVRD
Iona Island and Lions Gate WWTP
Project No. RFP 03-005

Appendix 10
Analysis of Preferred Options

FINAL REPORT

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August 2005

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1 INTRODUCTION

The purpose of this Appendix 10 is to further describe the preferred treatment options which were selected in Appendices 3 and 4 and confirmed at Workshop # 3 held on January 19, 2004. The planning process to select the preferred treatment options for interim upgrades and build-out to secondary was carried as follows:

- All treatment options for interim upgrades and for build-out to secondary were identified and are described in Section 7 of Appendix 3 and Section 4 of Appendix 4 respectively.

- A first level of screening was applied to all options specified in Appendix 3 and Appendix 4. The first level of screening consisted of a two-step process which initially included the application of pass or fail criteria. Processes that passed all criteria were further evaluated and the number of options for interim and build-out to secondary was reduced to approximately 5 options for each plant.

- The options that passed the first level of screening were evaluated in more details. The detailed analysis of these options is described in Section 9 of Appendix 3 for interim upgrades and Section 8 of Appendix 4 for build-out to secondary.

- Following the analysis of the options that passed the first level of screening, a second level of screening was carried out to select a short list of preferred options. The results of the second level of screening were reviewed at Workshop # 3 and are described in Section 10 of Appendix 3 and Section 9 of Appendix 4.

Subsequent to the selection of the short list of preferred options, a number of activities were carried out to further detail the preferred options. These activities are documented in this Appendix, except for item 1 which is included in Section 4 of Appendix 3.

1. Revised flow and load projections based on the Design Case scenario. This work is covered in Appendix 3.

2. Evaluation of the capacity of the current primary plants and forecast of effluent quality with respect to BOD and TSS concentrations. As part of this activity for the Iona Island Wastewater Treatment Plant (IIWWTP), additional interim treatment options to meet permit requirements were identified.

3. Some components for the interim upgrades and build-out to secondary were modified following review comments by GVRD. For example, providing additional primary sedimentation tanks (PST) for wet weather flow was deleted from the scope of work at IIWWTP and Lions Gate Wastewater Treatment Plant (LGWWTP) given there are no wet weather effluent quality issues.

4. Revised process design modeling was carried out based on revised loads and flow projections, and the deletion of additional primary sedimentation tanks. Results in Appendix 3 and Appendix 4 were based on standardized design parameters for long term planning. In this Appendix design parameters have been amended to more accurately reflect actual plant performance for the interim period.

5. Based on the revised process modeling and review comments from the GVRD, the size of the unit processes as well as the number of units/tanks was updated.

6. Revised capital and operating and maintenance cost estimates were prepared for the short list of preferred options.

7. A staging plan is proposed based on the forecast of effluent BOD and TSS concentrations.

8. Based on the revised process modeling, the sludge production projections were also revised.

The results of the analysis of the short list of preferred options which was carried out as part of this Appendix 10 was incorporated into the conceptual site plans which are included under a separate cover.

The short list of preferred process options is summarized in Table 1.1:

TABLE 1.1
SHORT LIST OF PREFERRED UPGRADE OPTIONS

Objectives	Iona Island WWTP Option	Lions Gate WWTP Option
Interim Upgrades (To 2021 Iona Island and 2031 Lions Gate)	1. RTF for 25% of ADWF 2. RTF for 50% of ADWF 3. CEP 4. CEP + RTF for 50% of ADWF with no secondary clarifiers	1. CEP 2. BAF for 50% of ADWF 3. CEP + BAF for 50% of ADWF
Build-out to Secondary (To 2036 Iona Island and 2046 Lions Gate)	1. TF/SC 2. BAF	1. BAF

Notes:
RTF: roughing trickling filter
BAF: biological aerated filter
CEP: chemically enhanced primary
TF/SC: trickling filter/solids contact
ADWF: average dry weather flow

2 SUMMARY OF FLOWS AND LOADS

2.1 IONA ISLAND

A number of factors were taken into account in order to establish the upper and lower envelopes for flows and loading in the Vancouver Sewage Area (VSA). In addition a design case was added to the flow and loads projections since it is unlikely that all the assumptions used to establish the upper and lower envelopes would occur at the same time.

Population

Population forecasts are one the basic components to establish sewage flows and load forecast and are shown in Table 2.1.

TABLE 2.1
POPULATION SCENARIOS IN VSA

Year	Lower Envelope	Upper Envelope	Design Case Scenario
2001– Census	616,379		
2021	700,000	750,000	740,000
2036	710,000	775,000	762,000
2051	720,000	800,000	784,000

Impact of Water Conservation Programs

In conjunction with the variability in population growth, the other significant factor in estimating future flows is the impact of water conservations measures. The impact of the existing water conservation program is taken into account for the most probable upper case scenario while the upper envelope assumes that per capita sewage generation rates would remain mostly unchanged. The per capita flows are summarized in Table 2.2.

TABLE 2.2
RESIDENTIAL AND COMMERCIAL PER CAPITA FLOWS IN VSA

Year	Lower Envelope (L/c/d)	Upper Envelope (L/c/d)	Design Case Scenario (L/c/d)
2001 – Existing All sources	➢ Residential (Res.): 270 ➢ Commercial (Com.): 166		
2021	➢ Res.: 214 ➢ Com: 153	➢ Res.: 264 ➢ Com: 166	➢ Res.: 220 ➢ Com: 166
2036	➢ Res.: 175 ➢ Com: 144	➢ Res.: 264 ➢ Com: 166	➢ Res.: 188 ➢ Com: 166

BOD and TSS Loading

The existing BOD and TSS contributions from the various sectors are shown schematically in Figures 2.1 and 2.2, including the residential, industry, trucked liquid waste (TLW), commercial & institutional (C&I), and surface runoff. Water conservation measures will have no impact on loading. The only variable regarding loading for the residential and the commercial and institutional (C&I) sectors is the contribution from the food garburators. Tables 2.3 to 2.6 summarize the loadings from the various sectors.

FIGURE 2.1
BOD LOADING (2002) IN VSA

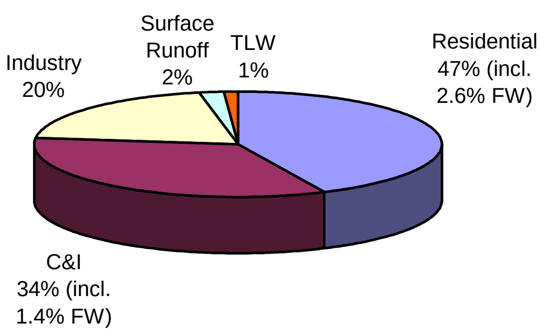
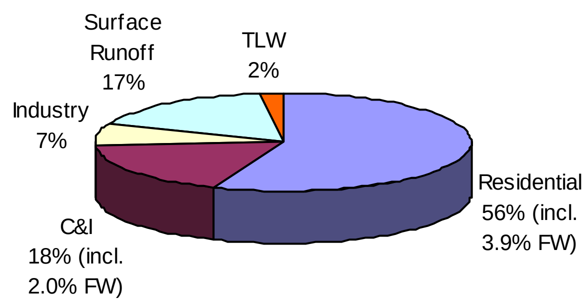


FIGURE 2.2
TSS LOADING (2002) IN VSA



Total BOD (AA) = 74.5 tonnes/day

Total TSS (AA) = 70.0 tonnes/day

Note: Food waste (FW)

TABLE 2.3
RESIDENTIAL AND C&I BOD SOURCE CHARACTERISTICS
(ANNUAL AVERAGE) IN VSA

Year	Lower Envelope (g/c/d)	Upper Envelope (g/c/d)	Design Case Scenario (g/c/d)
2001 – Existing All sources	➢ Residential (Res.): 53 ➢ Commercial (Com.): 41		
2021	➢ Res.: 52 ➢ Com: 39	➢ Res.: 54 ➢ Com: 41	➢ Res.: 54 ➢ Com: 41
2036	➢ Res.: 51 ➢ Com: 36.6	➢ Res.: 54.6 ➢ Com: 41	➢ Res.: 54.6 ➢ Com: 41

Note: Multiply all values by 1.31 to obtain the maximum month

TABLE 2.4
INDUSTRIAL, TLW AND SURFACE RUNOFF BOD (MAXIMUM MONTH) IN VSA

Year	Lower Envelope (t/d)	Upper Envelope (t/d)	Design Case Scenario (t/d)
2001 – Existing All sources	➢ Industrial (Ind.): 23.6 ➢ TLW: 2.1 ➢ Runoff: 1.8		
2021	➢ Ind.: 22.6 ➢ TLW: 2.2 ➢ Runoff: 1.8	➢ Ind.: 28.3 ➢ TLW: 2.5 ➢ Runoff: 1.9:	➢ Ind.: 27.6 ➢ TLW: 2.5 ➢ Runoff: 1.9
2036	➢ Ind.: 22.7 ➢ TLW: 2.2 ➢ Runoff: 1.8	➢ Ind.: 29.2 ➢ TLW: 2.6 ➢ Runoff: 2.0	➢ Ind.: 28.3 ➢ TLW: 2.6 ➢ Runoff: 2.0

TABLE 2.5
RESIDENTIAL AND C&I TSS (ANNUAL AVERAGE) IN VSA

Year	Lower Envelope (g/c/d)	Upper Envelope (g/c/d)	Design Case Scenario (g/c/d)
2001 – Existing All sources	➢ Residential (Res.): 61 g/c/d ➢ Commercial (Com.): 21 g/c/d		
2021	➢ Res.: 59 ➢ Com: 20	➢ Res.: 62 ➢ Com: 21	➢ Res.: 62 ➢ Com: 21
2036	➢ Res.: 59 ➢ Com: 19.6	➢ Res.: 63 ➢ Com: 21	➢ Res.: 63 ➢ Com: 21

Note: Multiply by 1.38 to obtain maximum month.

TABLE 2.6
INDUSTRIAL, TLW AND SURFACE WATER TSS CHARACTERISTICS (MAXIMUM MONTH) IN VSA

Year	Lower Envelope (t/d)	Upper Envelope (t/d)	Design Case Scenario (t/d)
2001 – Existing All sources	➢ Industrial (Ind.): 6.8 t/d ➢ TLW: 5.9 t/d ➢ Runoff: 15 t/d		
2021	➢ Industrial: 7.0 ➢ TLW: 5.9 ➢ Runoff: 15	➢ Industrial: 8.6 ➢ TLW: 7.1 ➢ Runoff: 16	➢ Industrial: 8.4 ➢ TLW: 7.0 ➢ Runoff: 16
2036	➢ Industrial: 7.3 ➢ TLW: 6.0 ➢ Runoff: 15	➢ Industrial: 8.9 ➢ TLW: 7.3 ➢ Runoff: 17	➢ Industrial: 8.6 ➢ TLW: 7.2 ➢ Runoff: 17

Projected Flows and Loads

Based on the data presented in Tables 2.1 to 2.6 above, flow and load projections for various scenarios are summarized in Table 2.7. The projected flows and loads in conjunction with the existing data for the period 1991-2002 are shown in Figures 2.3 to 2.5.

TABLE 2.7
IIWWTP FLOW AND LOAD SCENARIOS

	2021 – Design Year for Interim Upgrades			**2036 – Design Year for Build-out to Secondary**		
	Lower Envelope	Upper Envelope	Design Case	Lower Envelope	Upper Envelope	Design Case
ADWF (ML/d)	412	498	456	383	511	441
PWWF (ML/d)	1530	1530	1530	1530	1530	1530
Max Month BOD (t/d)	108	127	124	108	131	127
Max Month TSS (t/d)	106	120	116	105	124	119

The assumptions used when establishing the starting point for the loads and flow projections indicated above and shown on Figures 2.3 to 2.5 are:

Assumptions for Staring Point of Flow Projections:

- Per capita ADWF of 704 L/c/d (average of 1991-99)
- There are five years where the per capita ADWF is near 704 L/c/d (1993, 1995, 1996, 1998 and 1999)
- Population of 621,800 persons in 2002

Assumptions for Staring Point of BOD Load Projections

- Per capita loading of 0.125 g/c/d (upper envelope for period 1991-1999)
- There are four years where per capita loading varies between 0.120 and 0.125 g/c/d (1993, 1995, 1996, 1999 and 2001)
- Maximum month factor of 1.31 (average of period 1991-1999)
- Average of the maximum month factor for 2000-2002 is 1.29
- Starting point for projection: 0.125 g/c/d and max month factor of 1.31

Assumptions for Staring Point TSS Load Projections

- Per capita loading of 0.108 g/c/d (average of data for 1991-1999)
- There are seven years where per capital loading varies between 0.105 and 0.110 g/c/d (1996, 1997, 1998, 1999, 2000, 2001 and 2002)
- Maximum month factor of 1.38 (average of period 1991-1999)
- Average of the maximum month factor for 2000-2002 is 1.17
- Average of the maximum month factor for 1991-2002 is 1.33
- Starting point for projections: 0.108 g/c/d and max month factor of 1.38

FIGURE 2.3
IIWWTP FLOW PROJECTIONS (ADWF)

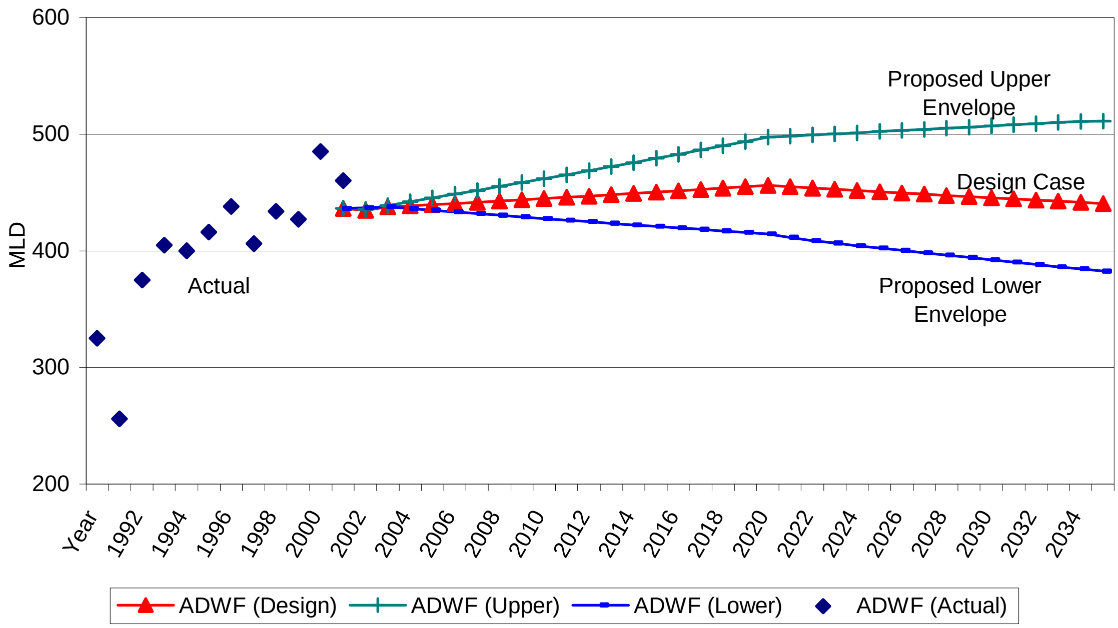


FIGURE 2.4
IIWWTP MAXIMUM MONTH BOD PROJECTIONS (TLW INCLUDED)

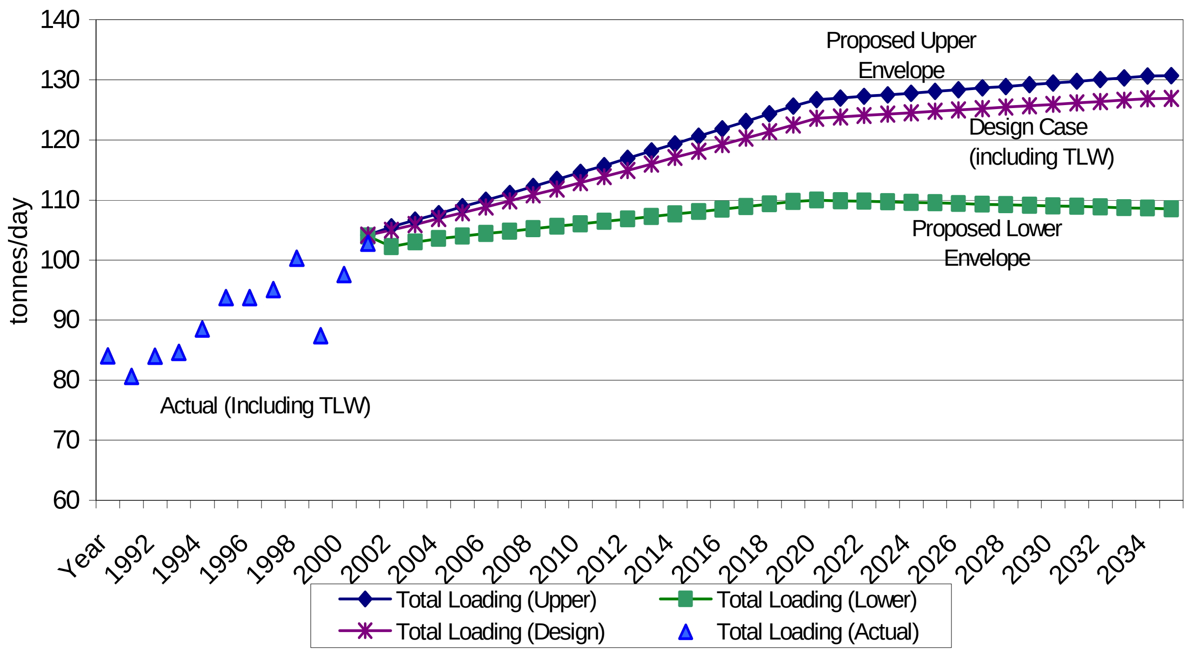
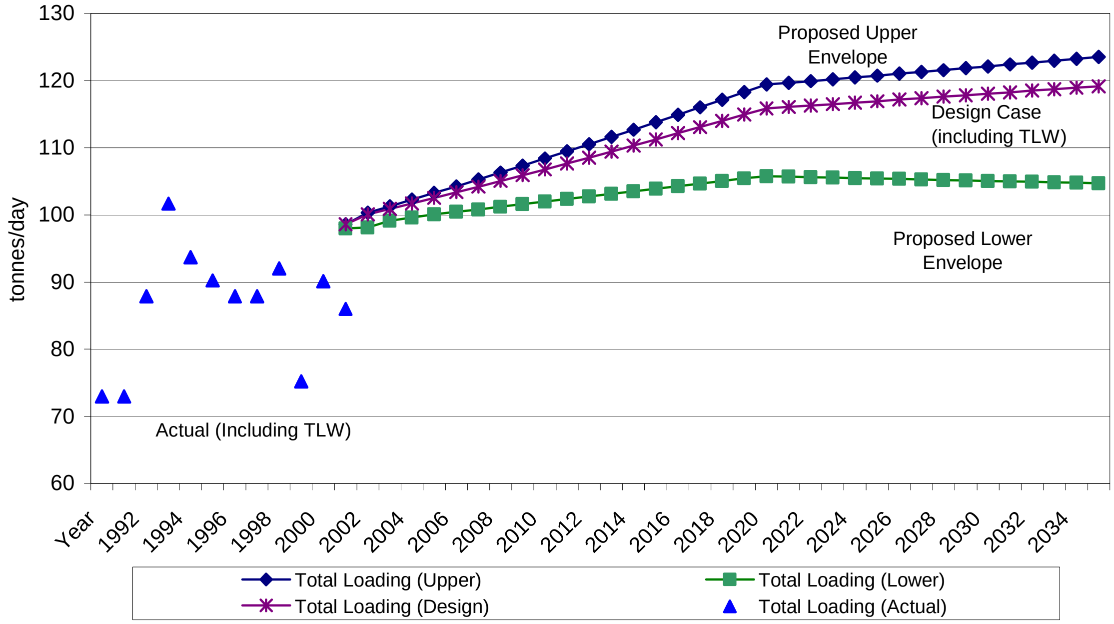


FIGURE 2.5
IIWWTP MAXIMUM MONTH TSS PROJECTIONS (TLW INCLUDED)



2.2 LIONS GATE

Similarly, a Design Case Scenario was established for LGWWTP for flow and load projections. The development of the Design Case scenario is based on the values in upper and lower envelopes established in Appendix 3 and Appendix 4.

Population

Table 2.8 summarizes the LGWWTP population scenarios (North Shore Sewage Area, NSSA). The upper and lower envelopes were established by the Regional Utility Planning division of the GVRD. The Design Case scenario is developed by lower envelope values plus 80% of the difference between the upper and lower envelopes.

TABLE 2.8
LGWWTP POPULATION SCENARIOS

Year	Lower Envelope	Upper Envelope	Design Case
2001 – Existing	173,750		
2031	215,000	244,000	237,000
2046	241,000	285,000	275,000
2051	250,000	300,000	289,000

Impact of Water Conservation Programs

In conjunction with the variability in population growth, the other significant factor in estimating flows is the impact of water conservation measures. The existing water conservation program is assumed for the Design Case scenario, while the lower envelope assumes "enhanced" water conservation initiatives. Upper envelope assumes per capita flow rate would remain mostly unchanged. Table 2.9 illustrates the per capita flow with these assumptions.

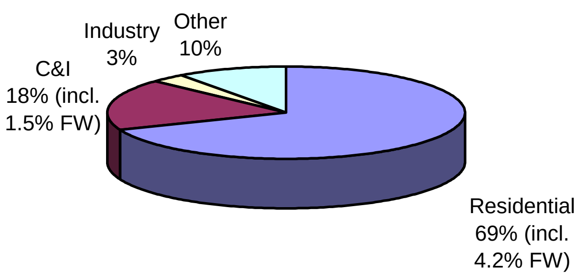
TABLE 2.9
RESIDENTIAL AND COMMERCIAL
PER CAPITA FLOW SCENARIOS (L/CAP/DAY) IN NSSA

Year	Lower Envelope	Upper Envelope	Design Case Scenario
2001 – Existing All sources	• Residential: 270 • Commercial: 55		
2021	• Residential: 232 • Commercial: 51	• Residential: 270 • Commercial: 55	• Residential: 243 • Commercial: 55
2036	• Residential: 232 • Commercial: 51	• Residential: 270 • Commercial: 55	• Residential: 243 • Commercial: 55

BOD and TSS Loading

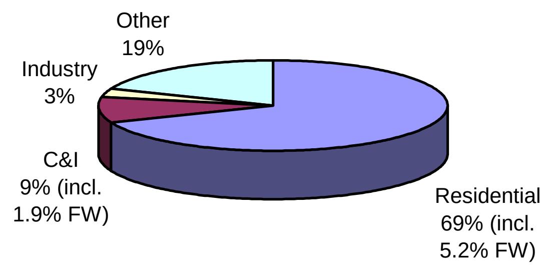
Figures 2.6 and 2.7 show the existing BOD and TSS contributions (2002) from various sectors based on the total average annual (AA) loading at the LGWWTP. Water conservation measures will have no impact on loading. The only factor that will impact the residential and commercial and institutional (C&I) contributions is the control on food waste discharge from garburators.

FIGURE 2.6
BOD LOADING (2002) IN NSSA



Total BOD (AA) = 12.3 tonnes/day

FIGURE 2.7
TSS LOADING (2002) IN NSSA



Total TSS (AA) = 14.8 tonnes/day

Note: Food waste (FW)

Tables 2.10 to 2.13 indicated the loading from various sectors that were used to generate the upper and lower envelopes and the Design Case.

TABLE 2.10
RESIDENTIAL AND C&I BOD SOURCE RUNOFF CHARACTERISTICS (ANNUAL AVERAGE) IN NSSA

Year	Lower Envelope	Upper Envelope	Design Case
2001 – Existing	• Residential: 53 g/c/d • Commercial: 14 g/c/d		
2031	• Residential: 52 • Commercial: 14	• Residential: 55 • Commercial: 15	• Residential: 53 • Commercial: 13
2046	• Residential: 50 • Commercial: 14	• Residential: 55 • Commercial: 15	• Residential: 54 • Commercial: 14

TABLE 2.11
INDUSTRIAL AND SURFACE RUNOFF BOD CHARACTERISTICS
(MAXIMUM MONTH) IN NSSA

Year	Lower Envelope	Upper Envelope	Design Case Scenario
2001 – Existing All sources	• Industrial: 0.5 t/d • Runoff: 1.9 t/d		
2031	• Industrial: 0.6 • Runoff: 1.9	• Industrial: 1.5 • Runoff: 2.0	• Industrial: 1.3 • Runoff: 2.0
2046	• Industrial: 0.6 • Runoff: 1.9	• Industrial: 1.8 • Runoff: 2.0	• Industrial: 1.6 • Runoff: 2.0

TABLE 2.12
RESIDENTIAL AND C&I TSS SOURCE RUNOFF CHARACTERISTICS
(ANNUAL AVERAGE) IN NSSA

Year	Lower Envelope	Upper Envelope	Design Case Scenario
2001 – Existing All sources	• Residential (Res.): 61 g/c/d • Commercial (Com.): 8 g/c/d		
2031	• Res: 58 • Com: 6	• Res: 63 • Com: 8	• Res: 62 • Com: 8
2046	• Res: 58 • Com: 6	• Res: 64 • Com: 8	• Res: 61 • Com: 8

Note: Multiply by 1.38 to obtain maximum month.

TABLE 2.13
INDUSTRIAL AND SURFACE RUNOFF TSS CHARACTERISTICS
(MAXIMUM MONTH) IN NSSA

Year	Lower Envelope	Upper Envelope	Design Case Scenario
2001 – Existing All sources	• Industrial: 1.0 t/d • Runoff: 4.0 t/d		
2031	• Industrial: 1.0 • Runoff: 4	• Industrial: 1.5 • Runoff: 4	• Industrial: 1.0 • Runoff: 4
2046	• Industrial: 1.0 • Runoff: 4	• Industrial: 1.8 • Runoff: 4	• Industrial: 1.2 • Runoff: 4

Projected Flows and Loads

Based on the data presented in Tables 2.8 to 2.13 in this Section, flow and load projections for various scenarios for LGWWTP are summarized in Table 2.14.

TABLE 2.14
LGWWTP FLOW AND LOAD SCENARIOS

	Existing	2031 – Design Year for Interim Upgrades			2046 – Design Year for Build-out to Secondary		
		Lower Envelope	Upper Envelope	Design Case	Lower Envelope	Upper Envelope	Design Case
ADWF (ML/d)	91	90	116	104	91	131	111
Peak Flow* (ML/d)	307	297	378	337	297	420	356
Max Month BOD (t/d)	18	21	26	25	23	30	28
Max Month TSS (t/d)	22	25	31	28	27	36	32

*: The GVRD's commitment in the Liquid Waste Management Plant (LWMP) is to treat 2×ADWF. The intension is therefore to manage I&I and wastewater flows to limit the peak flow to approximately 2×ADWF. The valves shown in this table are therefore theoretical.

The following assumptions were used when establishing the starting point for the flow and load projections indicated in Table 2.14 and Figures 2.8 to 2.10 for LGWWTP:

Assumptions for Starting Point of Flow Projection

- Per capita ADWF of 518 L/c/d (average of 1991-99)
- There are four years where the per capita ADWF is near 518 L/c/d (1993, 1994, 1995 and 1998)
- Population of 175,036 persons in 2002

Assumptions for Starting Point of BOD Load Projection

- Per capita loading of 0.077 g/c/d (average data for period 1991-1999)
- There are three years where per capita loading varied between 0.75 and 0.85 c/d/d (1993, 1994 and 1997)
- Maximum month factor of 1.34 (average period 1991-99)
- Maximum month factor is 1.08 (average of period 2000-2002)
- Maximum month factor is 1.12 (average of period 1991-2002)
- Start point for projection is 0.077 g/c/d multiplied by max month factor of 1.34

Assumptions for Starting Point TSS Load Projection

- Average annual per capita loading of 0.088 g/c/d (average data for 1991-1999)
- Three years where per capita loading between 0.08 and 0.09 g/c/d (1993, 1994 and 1999)
- Maximum month factor is 1.43 (average of period 1991-99)
- Maximum month factor is 1.43 (average of period 2000-2002)
- Maximum month factor is 1.14 (average of period 1991-2002)
- Starting point for projections: 0.088 g/c/d multiplied by max month factor of 1.43

Figures 2.8, 2.9 and 2.10 illustrate the projected lower, upper envelopes and the design case for ADWF, BOD (maximum month) and TSS (maximum month) respectively in conjunction with the historical data for the period 1991 to 2003.

FIGURE 2.8
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASE FOR ADWF

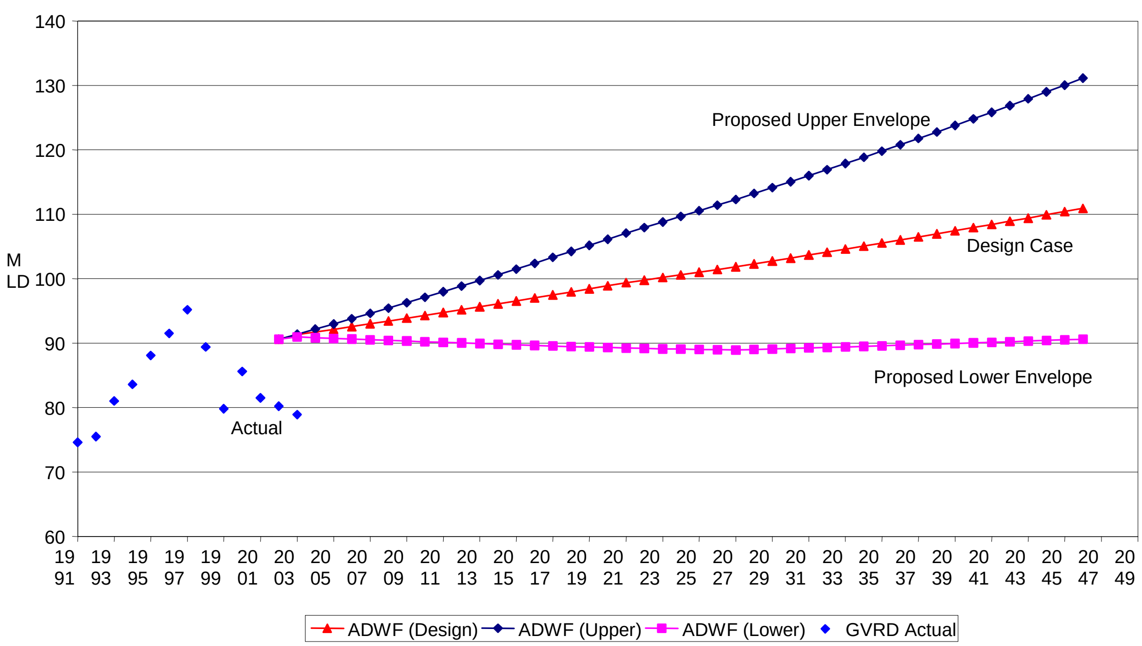


FIGURE 2.9
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASES FOR BOD (MAX. MONTH)

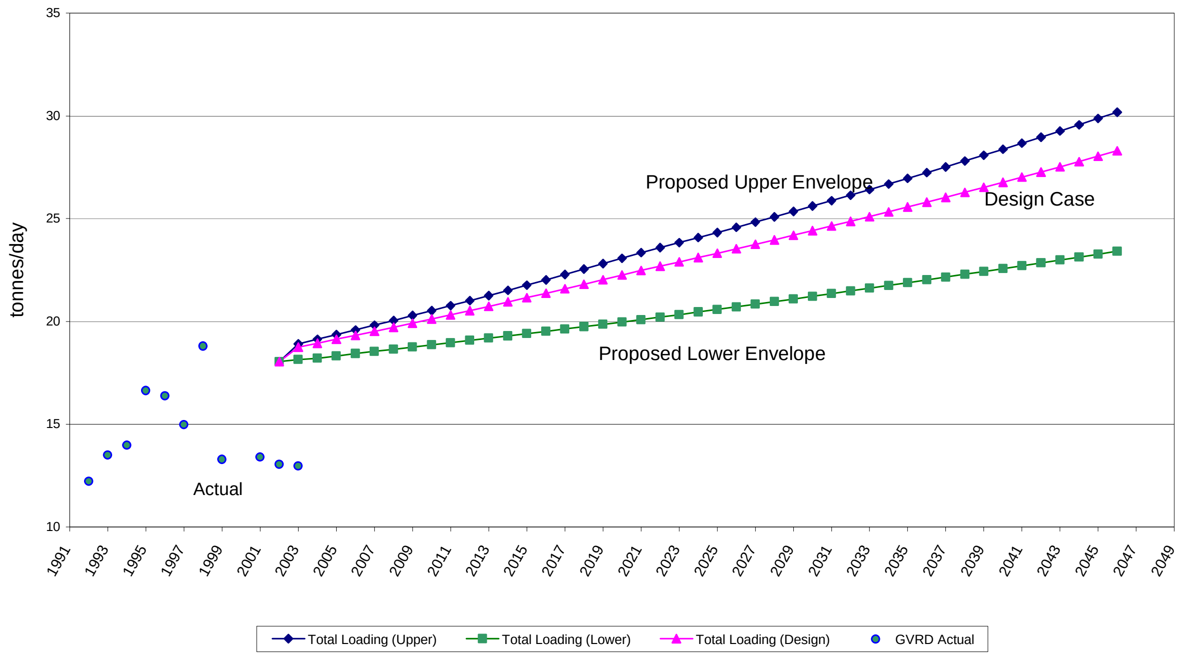
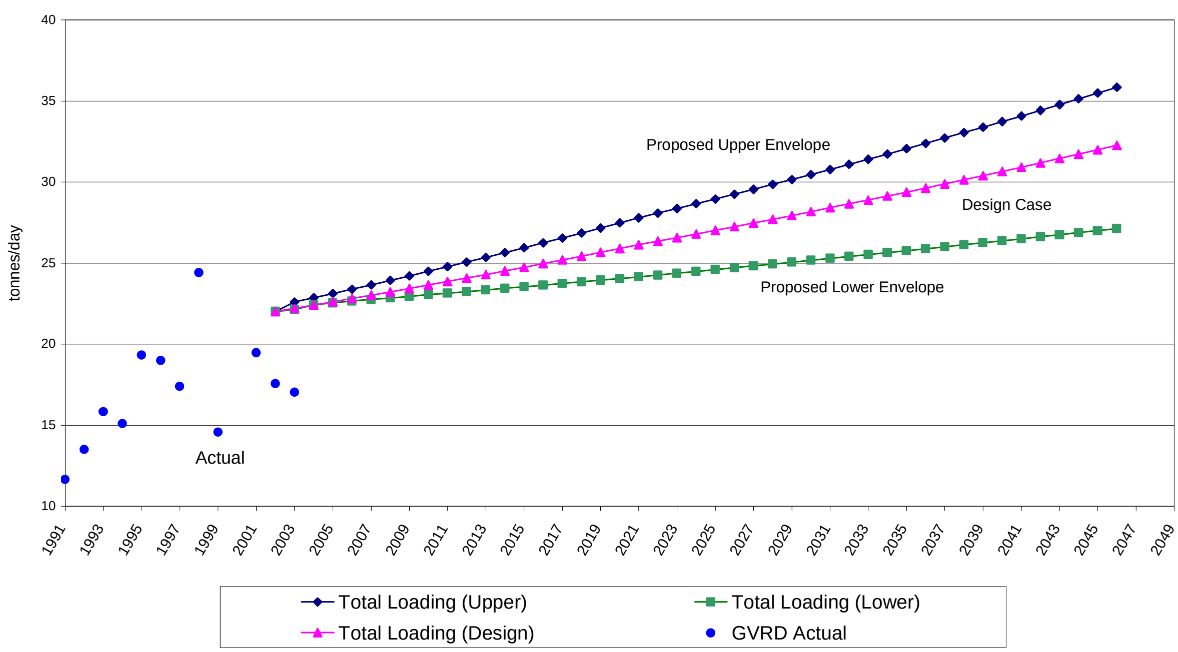


FIGURE 2.10
LGWWTP LOWER, UPPER ENVELOPES AND THE DESIGN CASE FOR TSS (MAX. MONTH)



3 FORECAST OF EFFLUENT QUALITY

3.1 IONA ISLAND

3.1.1 General

Primary sedimentation tanks (PST) are operated to remove substantial portions of readily settleable solids and organic substrates associated with solids. An efficient PST system is capable of removing 50~70% of total suspended solids (TSS) and 25~45% of biochemical oxygen demand (BOD), without additional chemical aids. However, the removal efficiency is subject to many factors, and many of these factors could be combined. The factors that influence PST performance are:

- Wastewater characteristics (e.g. solids settleability, organic content distributions)
- Surface overflow rate (SOR)
- Hydraulic retention time (HRT)
- Weir overflow loading (WOR)
- Sludge withdrawal rate
- PST configurations and tank geometry
- Eddy current of influent flow
- Wind induced circulation
- Thermal convention, density current, and thermal stratification

In addition, some specific operating conditions upstream of the PST will also affect the PST performance. At Iona Island these specific factors include flow distribution and influent pump operation. At the IIWWTP, the hydraulic factors (flow distribution, SOR and HRT etc.) and wastewater characteristics (settleable TSS and organic content distribution etc.) are considered to be the most important factors affecting the PST performance.

3.1.2 Primary Settling Tank Performance

The effluent quality for TSS and BOD concentrations for the past ten (10) years are shown in Figure 3.1 and Figure 3.2, respectively, against the percentile of occurrences. The effluent quality criteria of the maximum daily concentrations (flow proportioned-24 hr composite) are 130 mg/L of BOD_5 and 100 mg/L of TSS (Appendix C Operational Certificate ME0023). As a results of using all of the primary sedimentation tanks for wastewater treatment (prior to 1997, some PST were used only for TLW) the compliances level for effluent quality have improved substantially, from 90% (in the 1990s) to 99% (in early 2000s) of TSS and 88% (in the 1990s) to 98% (in early 2000s) of BOD, respectively, regardless of the flow and load increases through the years. Waste source control and water conservation measures in the VSA may have significant impacts on the sewage characteristics. In 2003 and 2004, no out of compliance at IIWWTP effluent was reported.

FIGURE 3.1
EFFLUENT TSS CONCENTRATION AT IIWWTP (1991~2002)

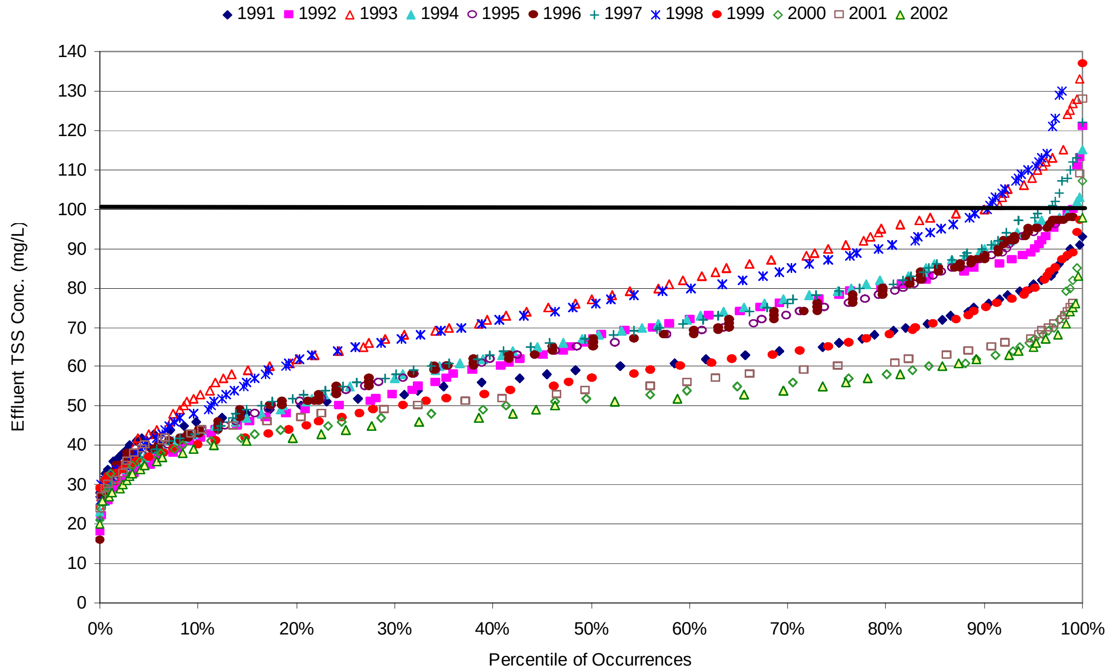
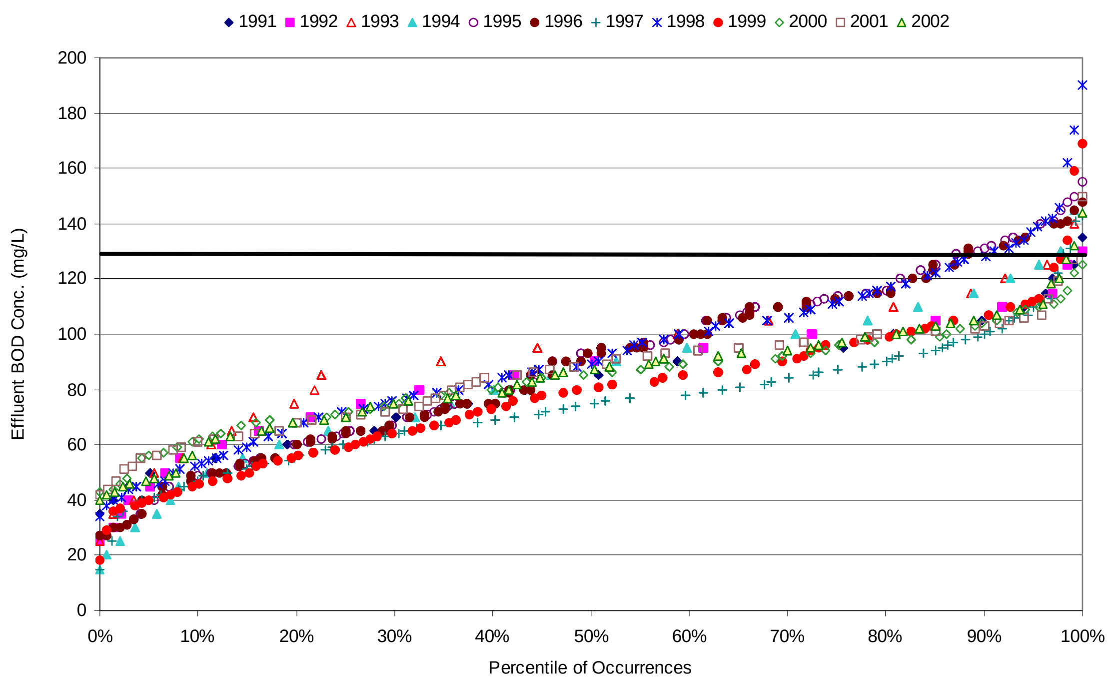


FIGURE 3.2
EFFLUENT BOD CONCENTRATION AT IIWWTP (1991~2002)



The SOR at average flow and peak flow conditions are usually the major criteria that govern the PST design and operation. The PST surface area requirement is governed by either the SOR at average flow or the SOR at peak flow. The PST at IIWWTP was originally sized using 45 $m^3/m^2/d$ (annual average flow, AAF) and 130 $m^3/m^2/d$ (peak wet weather flow, PWWF) as the design criteria. The typical design SORs at average and peak flow conditions (Metcalf & Eddy, 2003) are summarized in Table 3.1 for comparison.

TABLE 3.1
PRIMARY SEDIMENTATION TANK DESIGN CRITERIA
OF SURFACE OVERFLOW RATES (SOR)

Parameters	Unit	Metcalf & Eddy 2003*		IIWWTP	
		Typical	Range	Original Design Value	Recommended Design Value
SOR at Average Flow	$m^3/m^2/d$	40	30~50	45	50
SOR at Peak Flow	$m^3/m^2/d$	100	80~120	130	130
Detention Time	Hours	2	1.5~2.5	1.3 at average flow and 0.5 at peak flow	1.2 at average flow and 0.5 at peak flow

*: Metcalf & Eddy 2003, Wastewater Engineering, Treatment and Reuse, 4th edition, McGraw-Hill, Boston.

The correlations between the TSS/BOD removal efficiencies and the actual PST surface overflow rate are shown in Figures 3.3 and 3.4, respectively (complete 2001 daily flow rate proportional composite sample data). Typical PST removal efficiencies reported in Metcalf & Eddy (2003) are also shown in the figures for comparison. The removal efficiencies at IIWWTP were slightly higher than the typical PST averages within the ranges of operation with some exceptions. In Figures 3.5 and 3.6, similar trends were found that the removal efficiencies increased as the HRT increased with some exceptions.

FIGURE 3.3
PST SOR VS. TSS REMOVAL EFFICIENCY AT IIWWTP (2001)

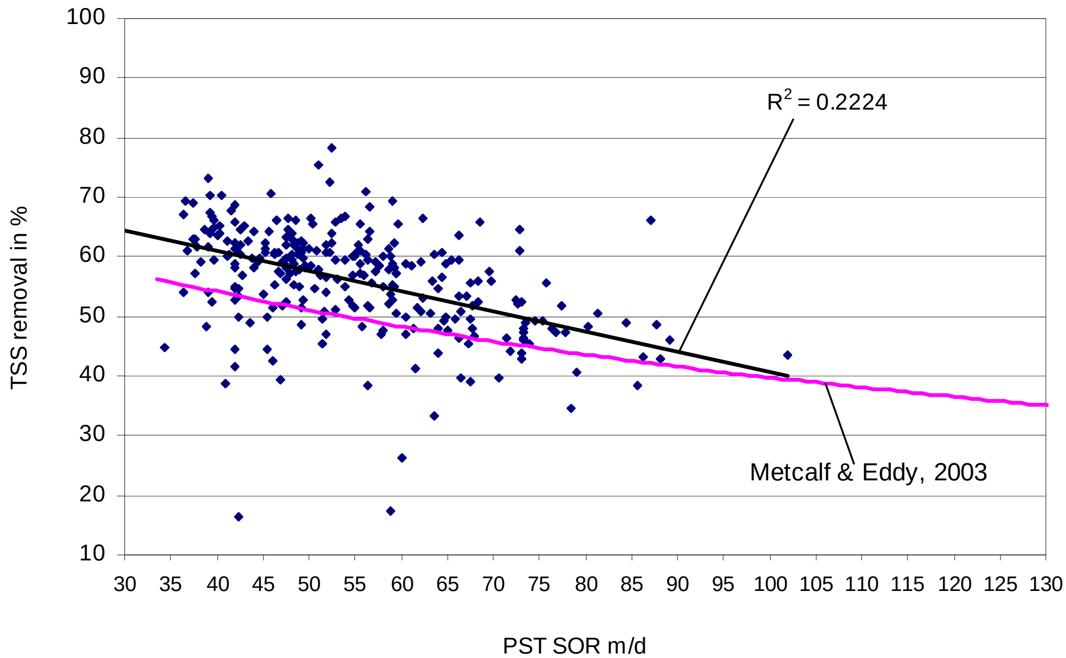


FIGURE 3.4
PST SOR VS. BOD REMOVAL EFFICIENCY AT IIWWTP (2001)

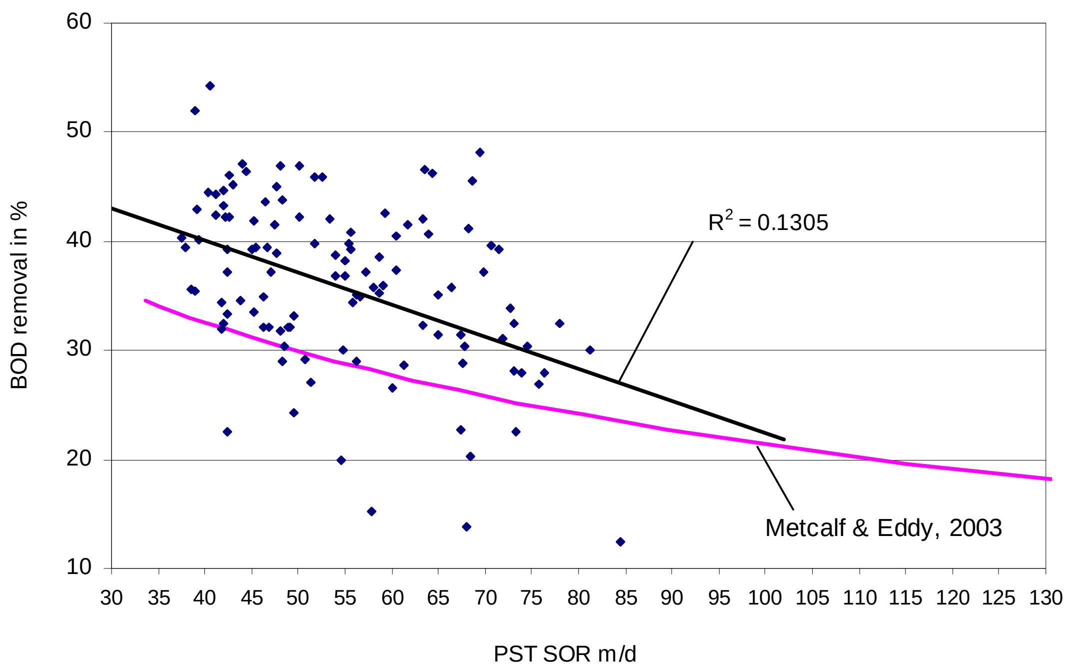


FIGURE 3.5
HRT IN PST VS. TSS REMOVAL EFFICIENCY AT IIWWTP (2001)

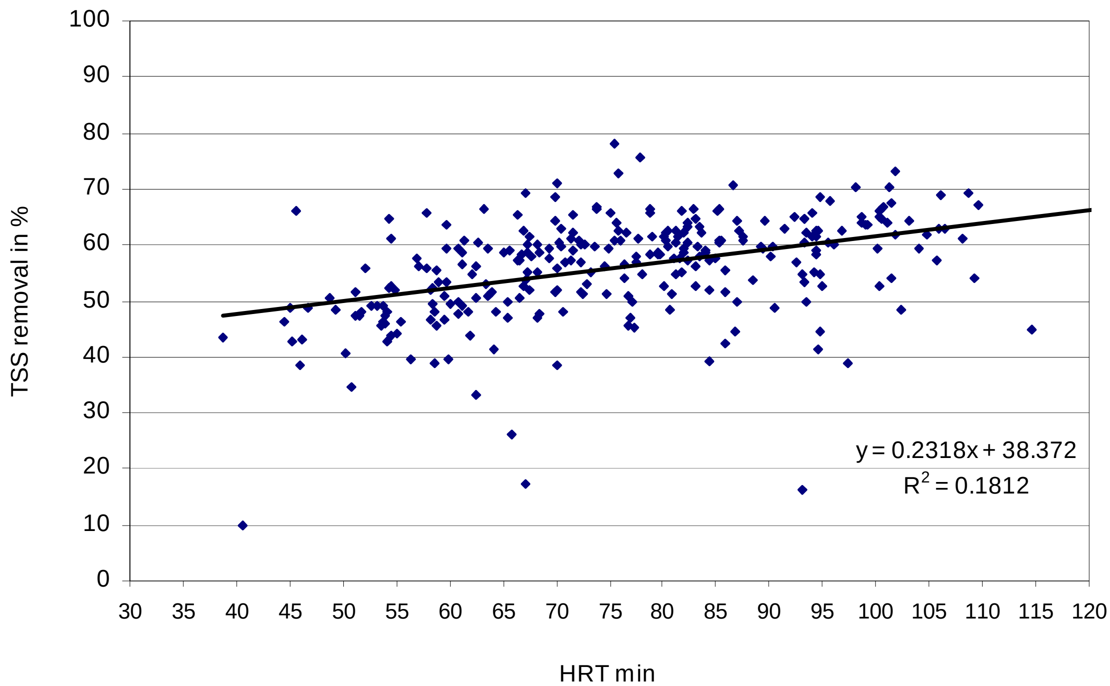
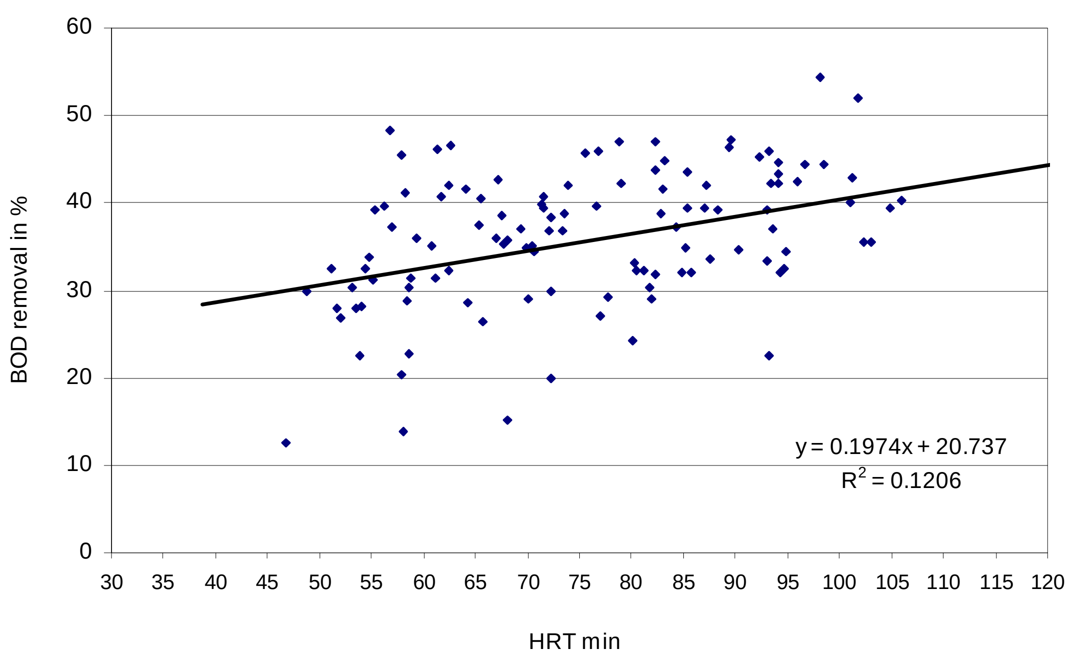


FIGURE 3.6
HRT IN PST VS. BOD REMOVAL EFFICIENCY AT IIWWTP (2001)



The TSS and BOD removal efficiencies at 45 $m^3/m^2/d$ SOR averaged about 60% (40~70%) and 38% (30~48%), respectively. With the design case flows of 593 ML/d (AAF) and 1,530 ML/d (PWWF), the existing PST will experience a SOR of 50 $m^3/m^2/d$ and 130 $m^3/m^2/d$ at average and peak flow conditions, respectively. The resulting removal efficiencies of TSS/BOD are estimated about 50%/35% at average flow, and 35%/15% at peak flow, respectively.

From the analysis of the plant data, it was found that the removal efficiencies at IIWWTP are best correlated to hydraulic factors, i.e. flow rate, SOR and HRT. This suggests that the PST performance appears to be governed by flows in general.

Several random exceptions of low removal efficiencies may be explained due to the variations of sewage characteristics. The GVRD has initiated efforts to investigate the potential influence of wastewater characteristics variations, which include the internal recycles (from dewatering lagoons), the ratio of SBOD/TBOD in the influent, and possible the first flush effect in the collection tributary (GVRD 2002 Quality Control Annual Report). It is also recommended to investigate the correlations between the wastewater characteristics and the rainfall record in the collection tributary (e.g. storm event in the VSA).

Several attempts were made to establish relationships between removal rates and other factors in order to determine if there are other significant factors regarding removal efficiencies based on the available information. The analysis of plant data carried out to determine if other factors would have a significant impact on PST removal rates include:

- Ratio of influent VSS/TSS vs. TSS removal efficiency (Figure 3.7) - There was no correlation between the ratio of VSS and TSS in the influent and the removal rates.

- Ratio of influent COD/VSS with BOD removal efficiency (Figure 3.8) – There was no correlation between the ratio of COD to VSS and the removal rates.

- Ratio of influent BOD/COD with BOD removal efficiency (Figure 3.9) – There was no correlation between the ratio of BOD to COD and the removal rates.

FIGURE 3.7
RATIO OF INFLUENT VSS/TSS VS. TSS REMOVAL EFFICIENCY AT IIWWTP (2001)

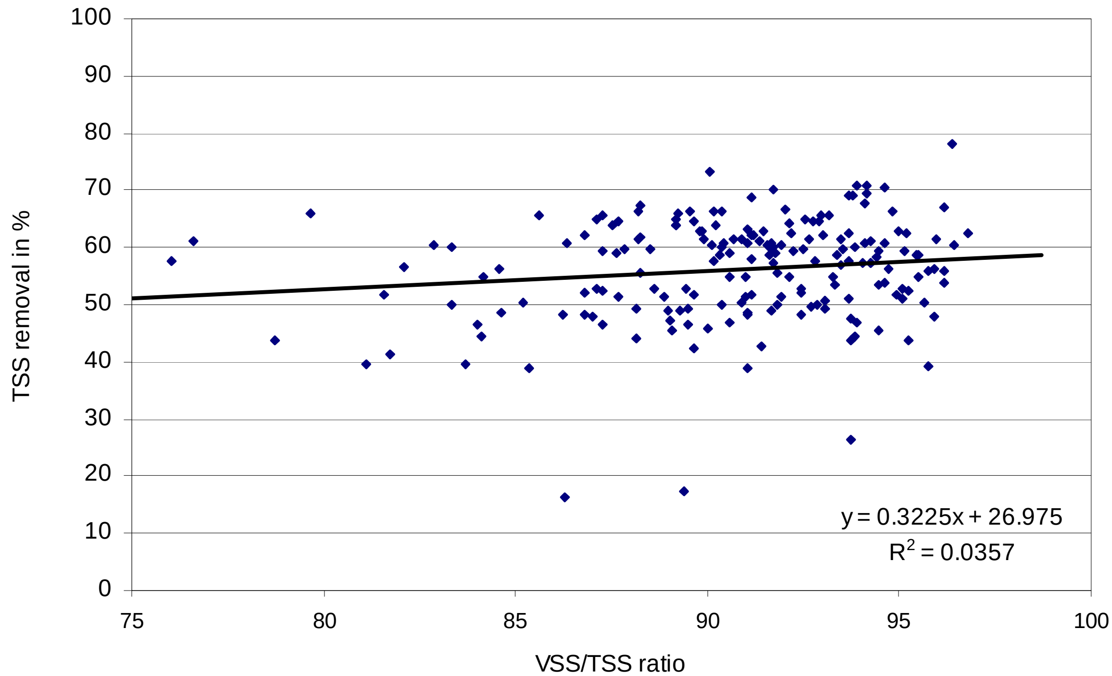


FIGURE 3.8
RATIO OF INFLUENT COD/VSS VS. BOD REMOVAL EFFICIENCY AT IIWWTP (2001)

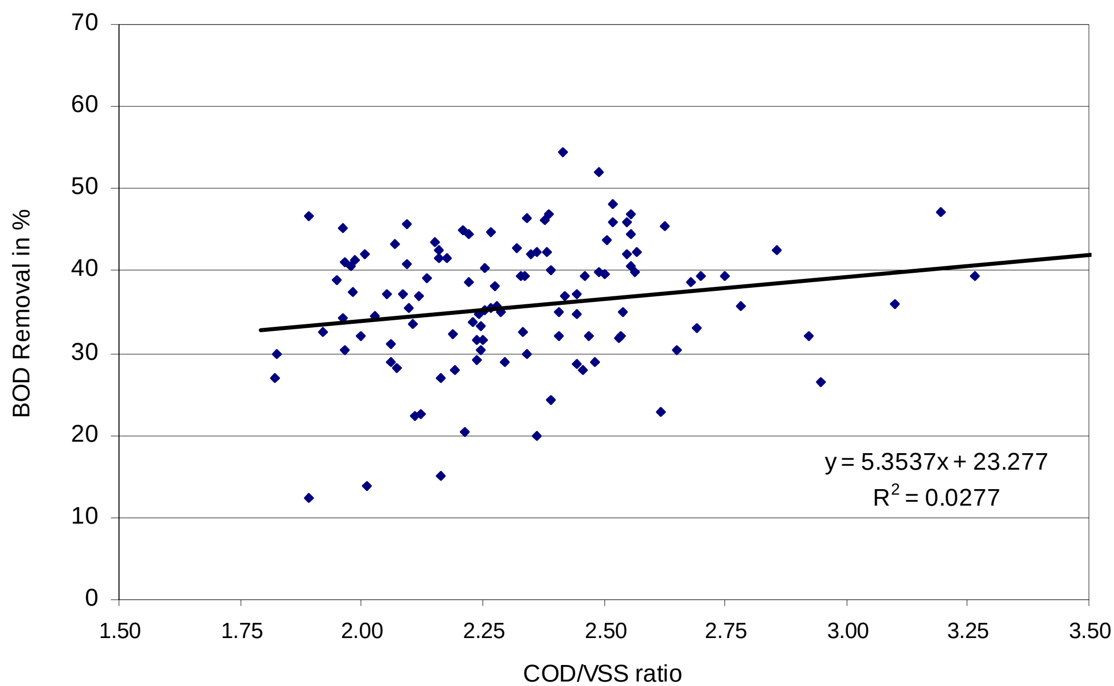
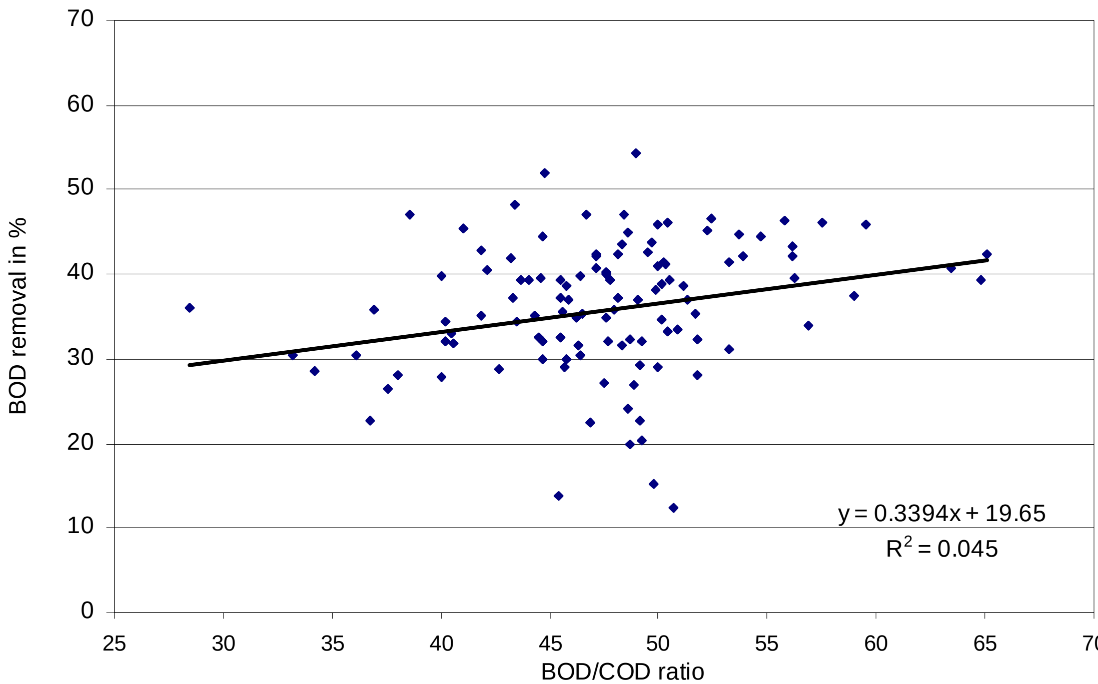


FIGURE 3.9
RATIO OF INFLUENT BOD/COD VS. BOD REMOVAL EFFICIENCY AT IIWWTP (2001)



3.1.3 <u>Proposed Design SOR</u>

The design strategies for eliminating PST capacity expansion are:

- Use the actual plant PST performance data to adjust the SOR criteria in design,
- Optimize the existing PST operation by improving flow distribution,
- Eliminate the need of PST capacity expansion given the need to target BOD removal,
- Adjust the secondary treatment capacity to accommodate the reductions of removal efficiencies due to higher SOR in PST if necessary and to target removal of soluble BOD,
- Meet the effluent criteria during interim and build-out to secondary.

The SORs of 40 $m^3/m^2/d$ at AAF and 100 $m^3/m^2/d$ at PWWF, which were initially selected in the analysis of various treatment options (Appendix 3 and Appendix 4), were valid for comparison purposes. At the pre-design stage using the design case with the selected processes (i.e. TF/SC and BAF), 50 $m^3/m^2/d$ at AAF and 130 $m^3/m^2/d$ at PWWF were used in the PST design to achieve 50% of TSS and 30% of BOD, respectively. It is recognized that at high flows the influent sewage is significantly diluted by stormwater. It is possible to eliminate the need for additional PST capacity and the plant can be operated with the existing PST capacity. This would reduce the costs of capital and O&M for plant expansion including additional PSTs, flow splitting, and conveying.

3.1.4 <u>Interim Effluent Quality And Permit Reliability Projections</u>

The projection of the effluent quality at IIWWTP is difficult due to the large number of factors such as wastewater characteristics, operating issues, and hydraulics. The forecast of effluent quality for comparison with the reliability is also challenging since the causes of non-compliance are not fully ascertained. In order to describe the possible scenarios during the interim stage to 2021, the consulting team has developed an approach using the effluent quality occurrence distribution for projecting the effluent quality (BOD and TSS composite concentrations).

Effluent concentrations in percentile of occurrence based on 2000~2002 average distribution patterns (Figures 3.1 and 3.2) and the projected increases in flows and loadings are used to project the future effluent quality in occurrence distributions. With respect to the effluent compliance regarding TSS concentrations, the average of 2000~2002 effluent concentration distributions (Figures 3.1 and 3.2) are used as the baselines for the effluent quality projections. It is assumed that the variation of wastewater characteristics and flow conditions are similar during the year, and the operating conditions (flow split, pump operation etc.) will remain the same as the current level. The projected annual average loads, annual average flow, and the PST removal efficiencies shown in Figures 3.3 and 3.4 are used to project the effluent quality in percentile of occurrence shown in Figures 3.10 and 3.11.

The methodology for this approach is summarized as follows:

- The 2000-2002 average of effluent concentration distribution shown in Figures 3.1 and 3.2 is used as a basis.
- Influent concentrations for BOD and TSS are calculated for future flows using average annual flows and loadings (50% percentile as the reference point).
- SOR for future flows are calculated and average removal efficiencies for each year are estimated using data in Figures 3.3 and 3.4.
- The percentile distribution of effluent quality occurrence is projected for the years 2004, 2011 and 2021.

With the PST plant without interim upgrade, the reliability to meet the discharge criteria for TSS is expected to be reduced from 99.5% (2004, 2 out of 365) to 98%, due to the increase of flow and loads. The reduction in effluent BOD reliability to meet the discharge criteria is estimated to be reduced from 97% to 80%.

It should be noted that the percentile of occurrence is one of the result-oriented approach to include all the probable factors in the sewage collection and treatment system. However, the percentile distribution may be affected by future hydraulic upgrade (flow split improvement), water conservation, waste source control, and possibly the storm events (first flush effect), which may result in the increase or decrease of the effluent exceedances. Additional monitoring of the PST performance as well as correlating the performance to the variations of wastewater characteristics (e.g. SBOD/BOD ratio, settleable solids/TSS ratio), hydraulic upgrade (flow distribution) and storm event in the tributary, may be used in the future to improve the projection accuracy.

FIGURE 3.10
IIWWTP PROJECTED EFFLUENT TSS CONCENTRATION IN PERCENTILE OF OCCURRENCE

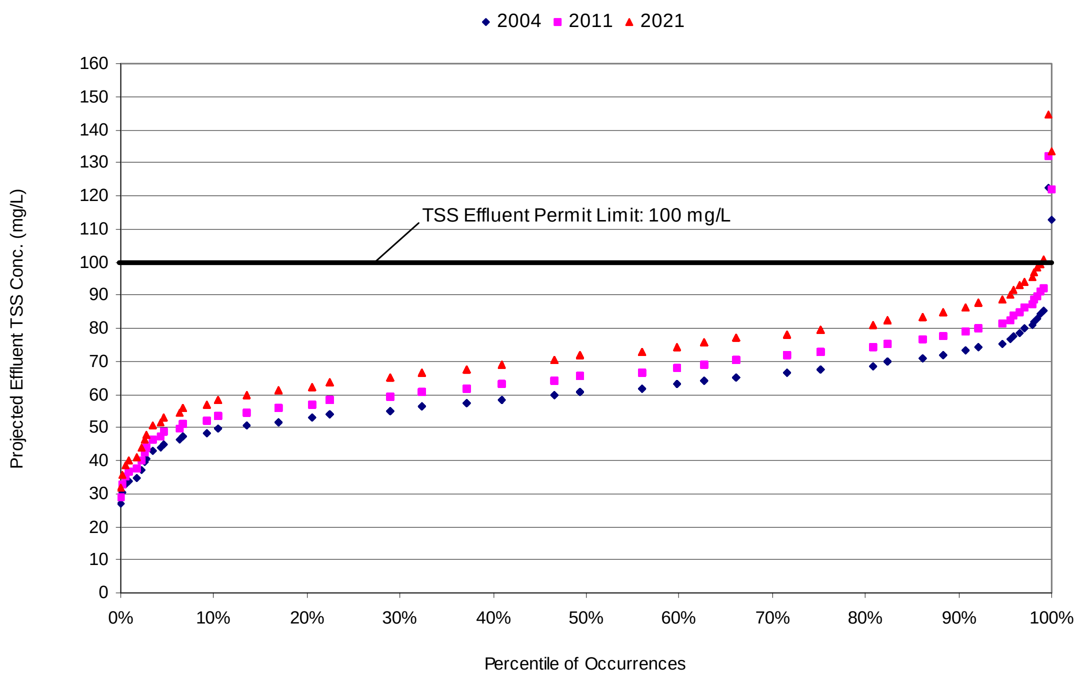
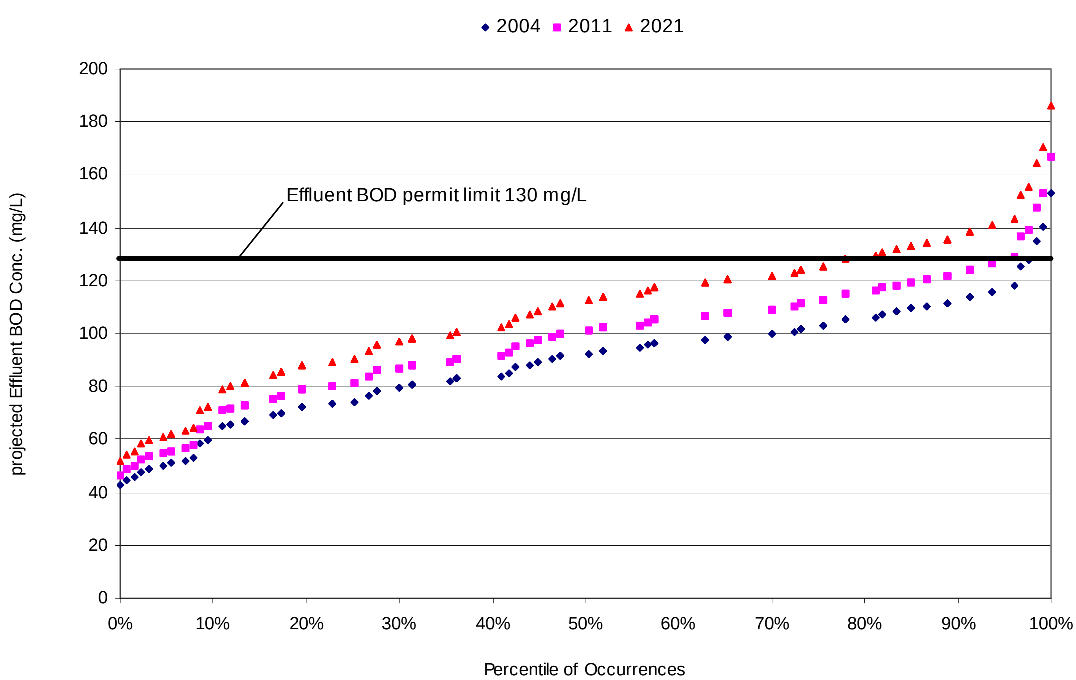


FIGURE 3.11
IIWWTP PROJECTED EFFLUENT BOD CONCENTRATION IN PERCENTILE OF OCCURRENCE



This approach is used to project the effluent quality of different interim upgrade options selected in Section 1 Table 1.1. The effluent quality reliability (against 100 mg/L of TSS and 130 mg/L of BOD) based on the effluent occurrence distribution projections are shown in Figure 3.12 (TSS) and 3.13 (BOD) respectively, in comparisons with a 99% reliability and the option with PST treatment only (no interim upgrade). The reliability of TSS concentration in effluent will be below 99% by 2009 without any interim upgrade. The effluent BOD concentration is already below 99% reliability at the current condition.

All four interim upgrade options could improve the effluent TSS and BOD reliability to meet the 99% target before 2021 (interim stage), except the interim Option 1 (RTF for 25% of ADWF) would fail to meet the BOD reliability of 99% by 2015.

FIGURE 3.12
PROJECTED EFFLUENT QUALITY RELIABILITY OF TSS CONCENTRATION AT IIWWTP

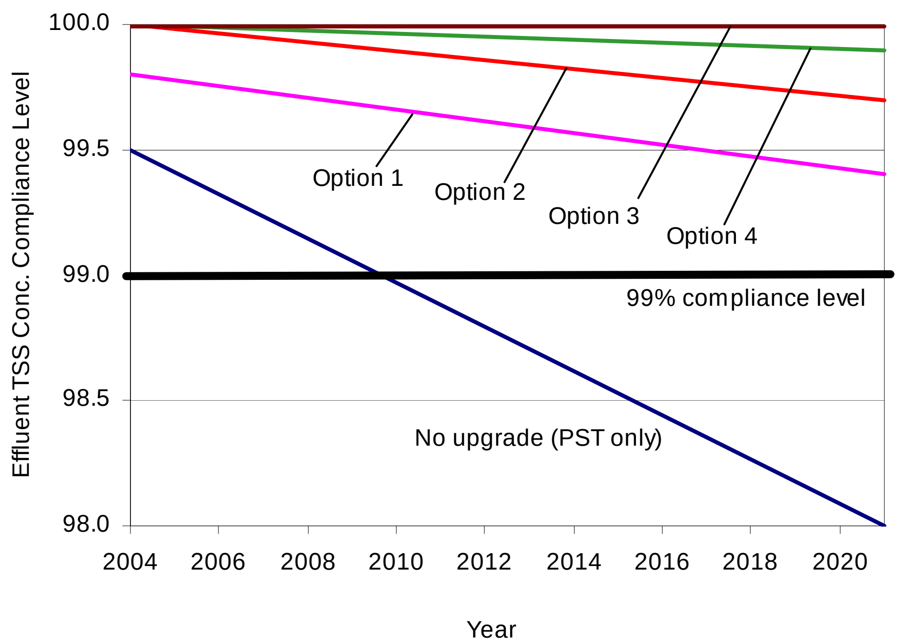
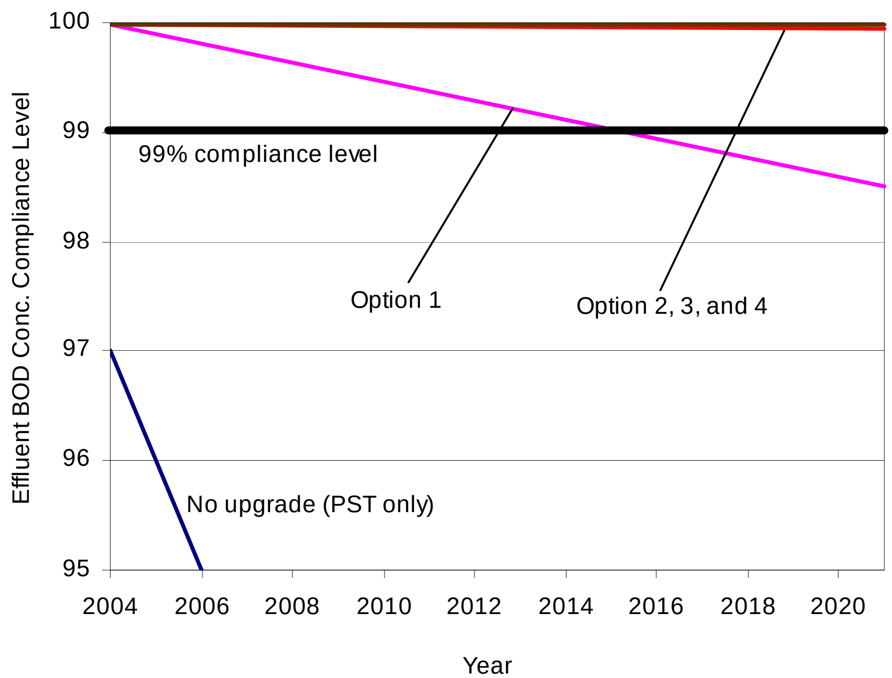


FIGURE 3.13
PROJECTED EFFLUENT QUALITY RELIABILITY OF BOD CONCENTRATION AT IIWWTP



3.2 LIONS GATE

3.2.1 General

General conditions of the PST performance and design consideration are described in Section 3.1.1. At the Lions Gate plant, the hydraulic factors (flow distribution, SOR and HRT etc.) and wastewater characteristics (settleable TSS and organic content distribution etc.) are considered to be the most important factors affecting the PST performance.

3.2.2 Primary Sedimentation Tank (PST) Performance

Because of the highly variable operating conditions of the primary settling tanks, assessing the performance can only be done on a statistical basis, using flow proportional composite daily data while reliability is measured on a per instance basis. Therefore only the extreme values of BOD and TSS are relevant to the understanding of the problem. Achieving compliance of a primary treatment plant under all conditions requires a very substantial factor of safety. Compliance can only be assessed in terms of the likely frequency of failure. Because the relationship of the measurable parameters, such as SOR and HRT, are well correlated to performance, it is not possible to define a single value which can indicate when a plant requires upgrading. At best a trend can be indicated and the probability of exceedance can be associated with that trend. This is the approach, which has been adopted here.

The effluent quality criteria of the maximum daily concentrations (flow proportioned-24 hr composite) are 130 mg/L of BOD_5 and TSS (Appendix C Operational Certificate ME0030). Figures 3.14 and 3.15 illustrate the distributions of effluent TSS and BOD concentrations (flow-proportional daily composites) respectively for the period of 1991 to 2003. Effluent TSS concentration complied comfortably with the Permit limit historically, whereas effluent BOD concentrations showed some exceedance above the limit in the year 1993, 1996, 1998 and 1999. Amongst which 1998 is the worst year. Since failures of less than 10% occurrence are of interest the analysis which follows focuses on these extreme values.

FIGURE 3.14
EFFLUENT TSS CONCENTRATION AT LGWWTP (1991-2003)

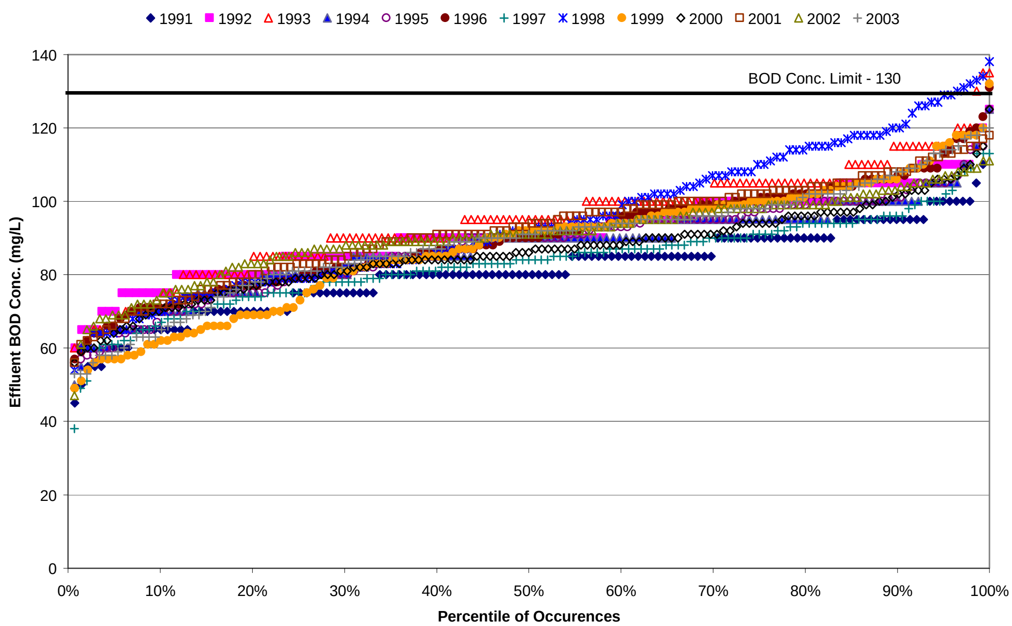
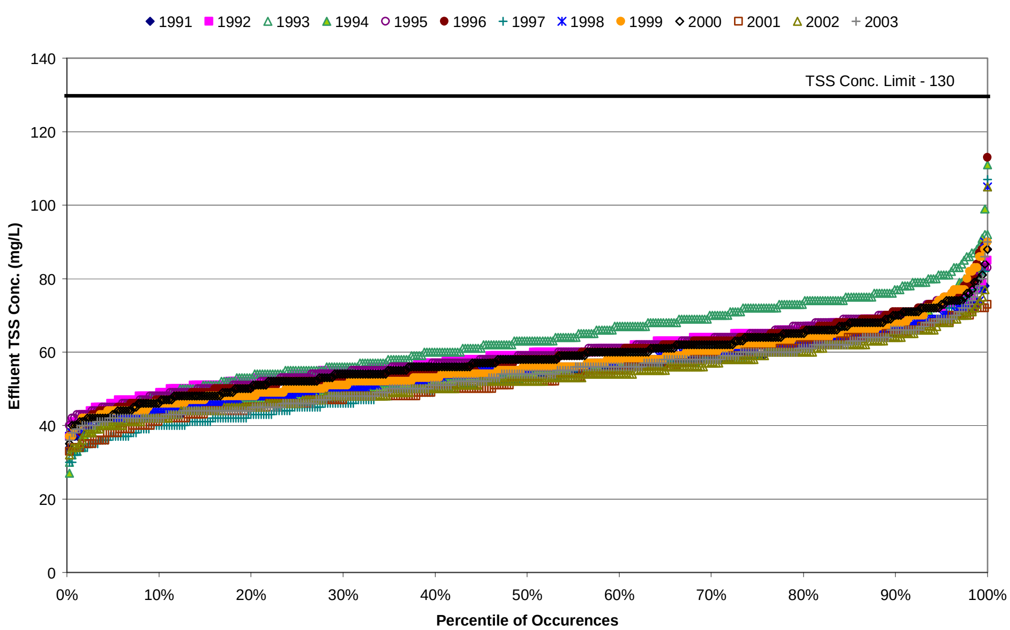


FIGURE 3.15
EFFLUENT BOD CONCENTRATION AT LGWWTP (1991-2003)



Figures 3.16 to 3.19 illustrate the variability of the data and the correlation between SOR and percent removal of TSS and BOD for the years 2001 and 2002. The trend to poorer removal with increasing average day SOR is apparent.

FIGURE 3.16
PST SOR VS. TSS REMOVAL EFFICIENCY AT LGWWTP (2001)

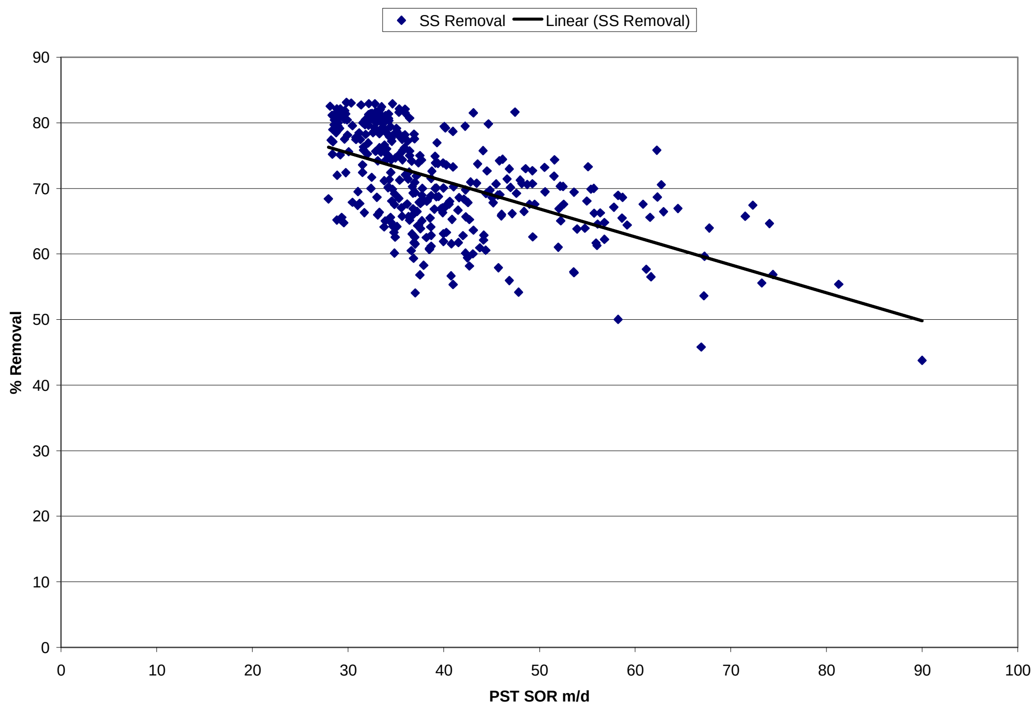


FIGURE 3.17
PST SOR VS. BOD REMOVAL EFFICIENCY AT LGWWTP (2001)

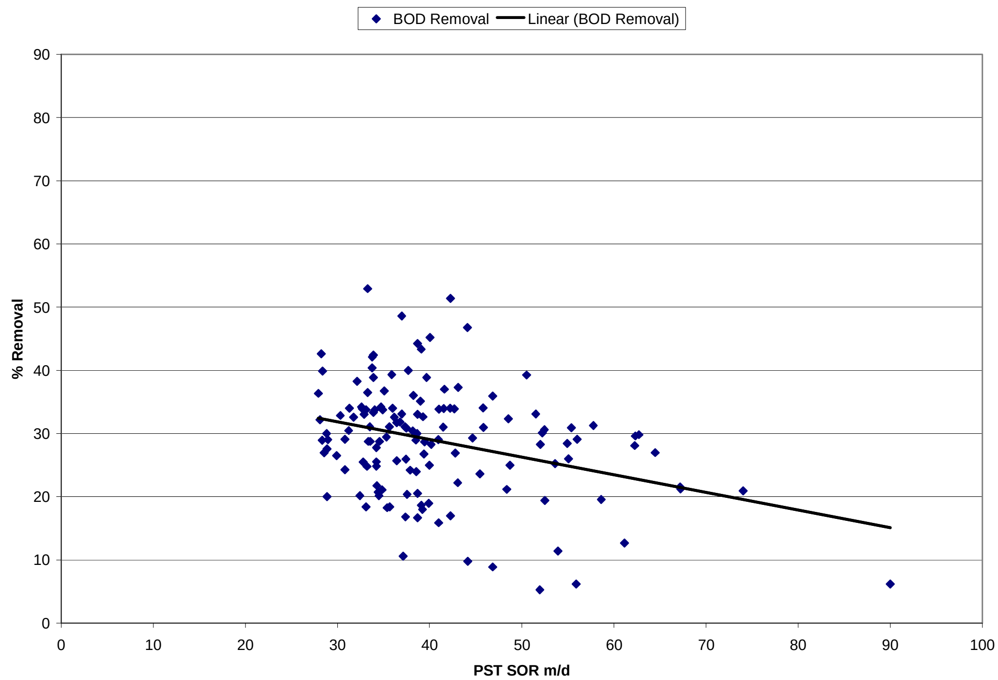


FIGURE 3.18
PST SOR vs. TSS REMOVAL EFFICIENCY AT LGWWTP (2002)

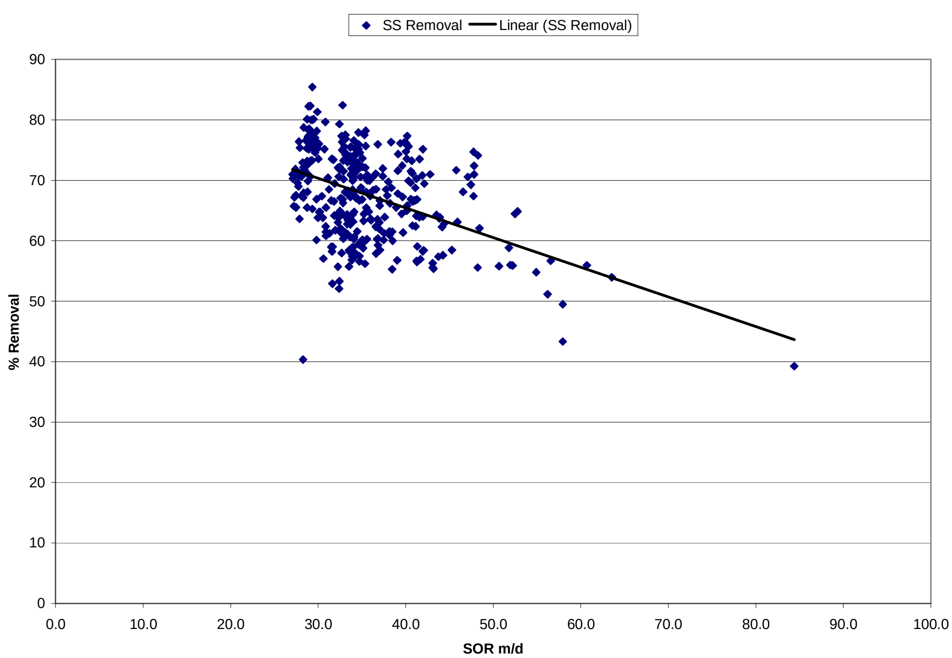
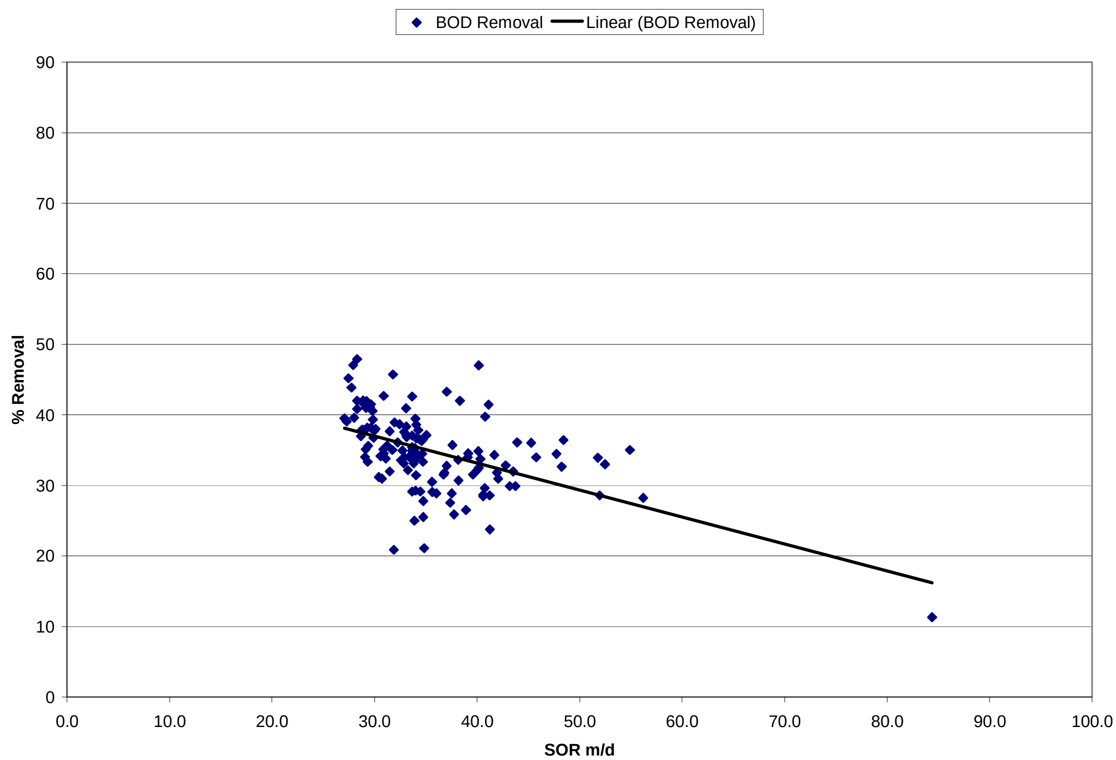


FIGURE 3.19
PST SOR vs. BOD REMOVAL EFFICIENCY AT LGWWTP (2002)



3.2.3 Forecast Effluent Quality

Figure 3.20 (TSS) and Figure 3.21 BOD illustrate the effluent quality percentile of occurrences for LGWWTP . The methodology used in developing these graphs is discussed in Section 3.1.4. These figures hold similar assumptions as Figure 3.12 and 3.13 developed for the Iona Island WWTP. Flow and load for the Design Cases are used in establishing projections for year 2004 and 2031.

FIGURE 3.20
LGWWTP PROJECTED EFFLUENT TSS CONCENTRATION IN PERCENTILE OF OCCURENCES

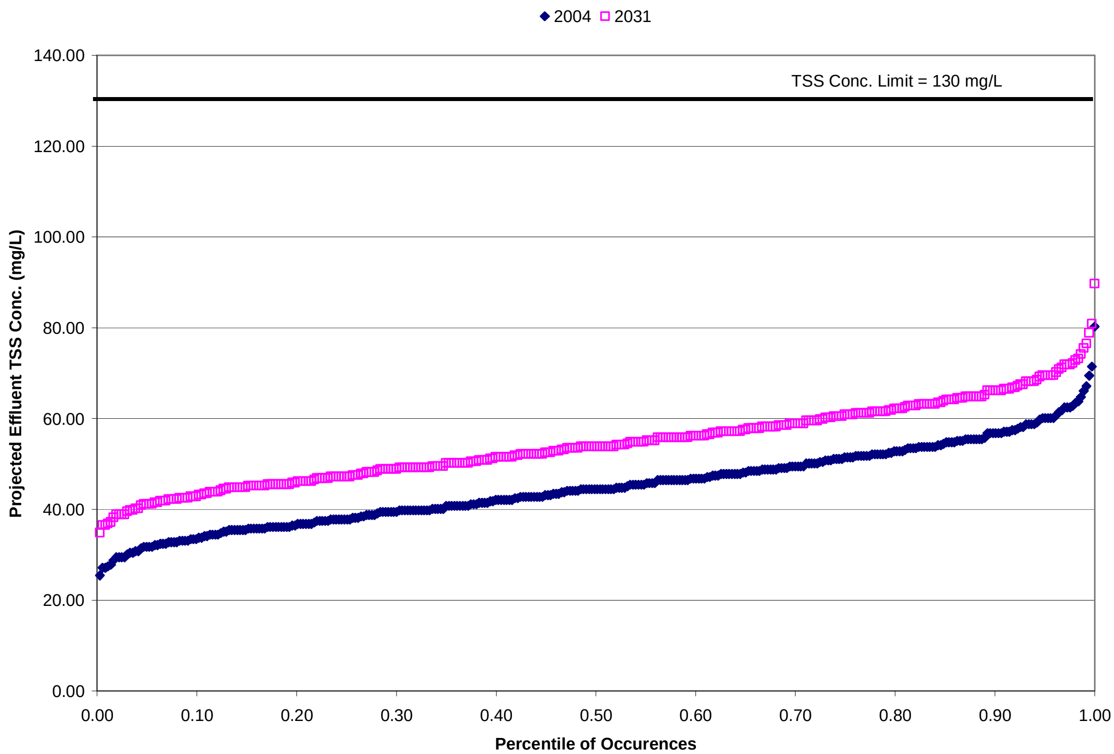
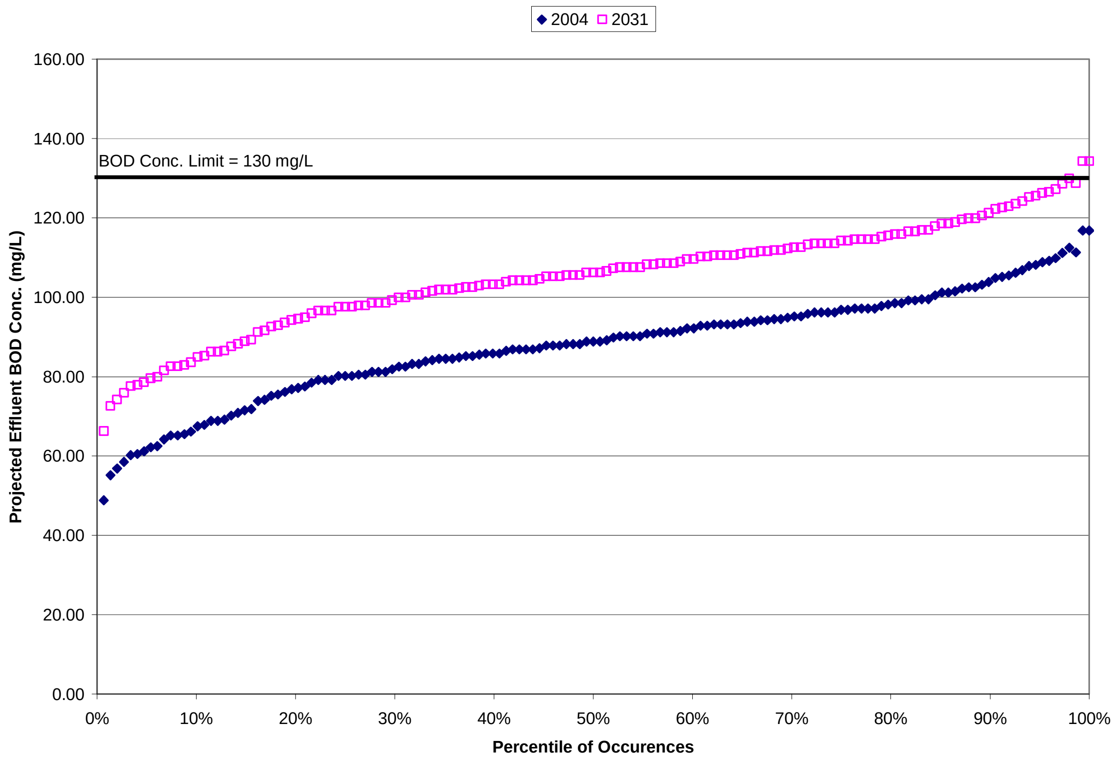


FIGURE 3.21
LGWWTP PROJECTED EFFLUENT BOD CONCENTRATION IN PERCENTILEOF OCCURRENCES



<u>Observations</u>

1. BOD is the critical parameter for compliance to 2031
2. The increase in BOD and TSS concentration throughout years is driven by two factors:
   - ➢ The increasing concentration in the influent to the plant as calculated from the independently derived flow and load scenarios. Success with water demand management and I&I reduction will negatively impact compliance of the plant with BOD concentration limits.
   - ➢ The increasing flows result in reduced efficiency of BOD and TSS removal. This effect is of lesser importance.
3. The accuracy of this forecast is dependent on the relative projection of both flow and load being accurate.
4. Demand management of load, by controlling the use of garburators may not impact this problem since the coarse material produced from the grinders may settle easily in the PSTs and not impact the BOD in the effluent.
5. Increasing the area of the PSTs, which have rather poor BOD removal efficiency (30% or less), appears to be an inefficient way to address the problem. Also, primary sedimentation tanks do not remove the soluble BOD

6. Under the Design Case, the SOR based on AAF increases from 40 m/d in 2003 to 45 m/d in 2030 and to 48.5 m/d in 2046.
7. For compliance with BOD concentration limits, design of PSTs based on the PWWF SOR is not rational since at this time concentration is diluted by the inflow and infiltration.
8. Centrate discharge, which is high in BOD, will impact negatively on reliability until treatment is introduced. Because sampling is composite and flow proportional, the impact of the discharge of centrate is included in the above data.

The projected reliability of effluent concentration for TSS and BOD for the preferred options for LGWWTP are illustrated in Figure 3.22 and 3.23 respectively. The Interim treatment options are:

- Option 1 – CEP only
- Option 2A – 50% ADWF BAF
- Option 2B – 50% ADWF BAF + CEP

The reliability projection from 2004 to 2030 for TSS easily achieves 100% at the limit of 130 mg/L for all options even if no upgrade is provided. Interim upgrade is required because the reliability of compliance with the BOD limit is projected to drops to below 99% in the year 2018.

FIGURE 3.22
LGWWTP PROJECTED RELIABILITY OF EFFLUENT CONCENTRATION FOR TSS

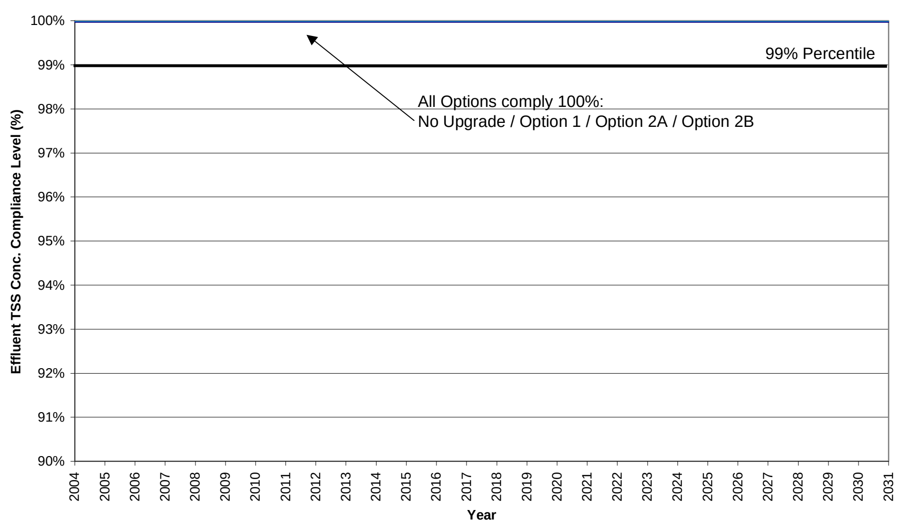
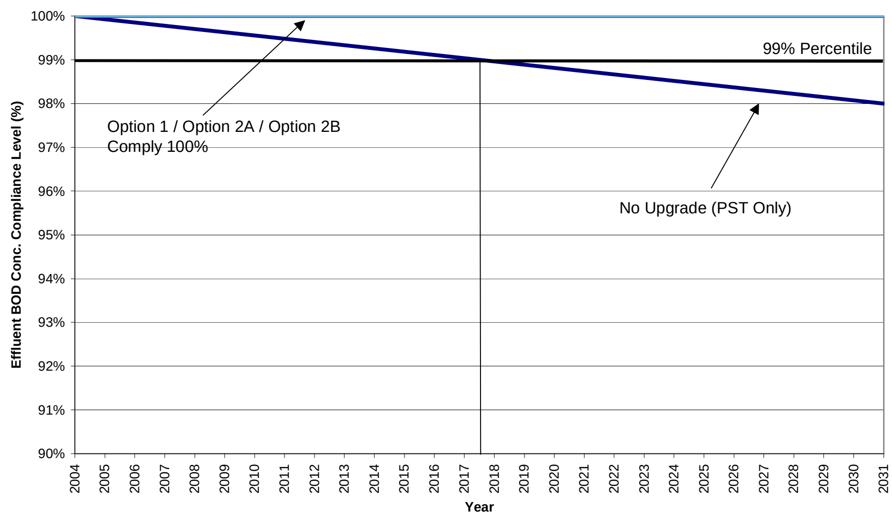


FIGURE 3.23
LGWWTP PROJECTED RELIABILITY OF EFFLUENT CONCENTRATION FOR BOD



3.2.4 Discussion

Under the scenarios developed in this report it would appear that reliability at the 99% level will become difficult to achieve after 2018 based on the design case for BOD concentration. Effectiveness of water demand management and I&I reduction appear to have the greatest negative impact on the BOD concentration in the effluent.

3.2.5 Conclusions

1. Some form of enhanced treatment is most probably required before the scheduled upgrading to secondary treatment in 2031.
2. BOD concentration will determine the date when enhanced treatment is required.
3. Increasing the size of the PSTs will not be a cost-effective means of increasing BOD removal.
4. Trending of the efficiency of the existing PSTs, the effectiveness of water demand management and of I&I control will identify when enhanced treatment will be required.
5. Investigation of the impact of centrate discharge on the BOD concentration could point to a partial solution.

4 ANALYSIS OF PREFERRED OPTIONS

4.1 IONA ISLAND

4.1.1 Description of Options for Build-Out to Secondary

Following the completion of Workshop # 3, the two preferred secondary treatment options for upgrading IIWWTP for build-out to secondary were selected, which include Trickling Filter/Solids Contact (TF/SC, Option 1) and Biological Aerated Filter (BAF, Option 2). TF/SC was selected as one of the preferred option on the following basis:

- Lowest life cycle cost (LCC)
- Lower operating and maintenance (O&M) cost than conventional activated sludge (CAS) or high rate activated sludge (HRAS)
- Compatibility with other secondary GVRD plants (Annacis Island, Lulu Island, and Northwest Langley WWTPs)
- Smaller footprint compared to CAS
- Ease of phasing and upgrading from RTF (for interim) to build-out to secondary.
- Lower energy consumption compared to CAS

Following the analysis of options for LGWWTP, biological aerated filter (BAF) was selected as another preferred option for build-out to secondary for the Iona Island WWTP. The BAF process was added to the short list of options for the for the following reasons:

- The cost of BAF is comparable to TF/SC
- BAF has the lowest footprint of all biological processes and this would allow for greater site utilization
- A smaller footprint would reduce site preparation (fill and pre-loading) and would reduce the need for hauling a large volume of dewatered sludge that is stored on site

The BAF process was initially not retained for Iona Island because of the lack of existing plants of similar size. However, following discussions at Workshop # 3, this option is added to the short list of preferred options for the reasons indicated above. Also since the use of this technology is increasing in the North America and worldwide, it is likely that plants comparable in size to IIWWTP will be built by the time the build-out to secondary will be required.

Build-out to Secondary Process Option 1 - Trickling Filter/Solids Contact (TF/SC)

The preliminary (screen and grit removal) and primary (primary sedimentation tank) treatment units will treat the entire flow collected from the Vancouver Sewage Area (VSA). No additional primary sedimentation tanks will be provided. The existing headworks will be upgraded and fine (6 mm) screening will be provided.

The TF/SC process is designed to provide a capacity of two times of the average dry weather flow (ADWF) for a hydraulic capacity of 912 ML/d (456 ML/d×2 = 912 ML/d), at 100% of the maximum month flow (MMF) loading of 83 t/d of BOD and 60 t/d of TSS following primary treatment. It should be noted that even though flows may decrease between 2021 and 2036 as a results of the water conservation measures already in place and sewer separation, the higher 2021 flow should be used for design.

Final clarifiers will be used to remove TSS and the biological sludge generated from the TF/SC process. The flow in excess of two times the ADWF will bypass the secondary treatment units (i.e. TF/SC) and will be discharged directly to the outfall pumping station after the primary treatment.

The primary sludge and biological sludge will be thickened in the gravity thickeners and dissolved air flotation (DAF) units, respectively. The thickened sludge from both streams will be mixed in a blending tank and further stabilized in the same anaerobic digesters to achieve volatile solids (VS) reduction and pathogen kill. The anaerobic digesters will be designed to be operated under mesophilic or thermophilic conditions, subject to the final biosolids product and/or land application requirements, e.g. Class A or Class B biosolids. The digested biosolids will be dewatered to reduce the volume before reuse/recycling. The rejected wastewater from the sludge handling processes (thickeners, digesters and dewatering units) is recycled back to the plant for treatment.

Build-out to Secondary Process Option 2 - Biological Aerated Filter (BAF)

The preliminary (screen and grit removal) and primary (PST) units are designed to treat the entire flow collected from the VSA. The BAF process will be designed to provide a capacity of two times of the ADWF for a hydraulic capacity of 912 ML/d (456 ML/d×2 = 912 ML/d), at 100% of the maximum month flow (MMF) loading of 83 t/d of BOD and 60 t/d of TSS following primary treatment. The BAF process does not require final clarifiers to remove the TSS and biological sludge generated in the biological process, rather, the solids are removed in the BAF back wash cycles. The flow in excess of two times of the ADWF will bypass the secondary treatment (i.e. BAF) units and discharge directly to the outfall pumping station after the primary treatment. Solids handling would be the same as described in the TF/SC option.

Since BAF was added as an additional option later in the study, this option was not examined to the same level of detail as TF/SC regarding the preparation of concept drawings. The biological aerated filter was sized based on using the Biofor® system as supplied by Degremont®. A layout plan using the Biofor® BAF process was prepared mainly to allow a comparison of site utilization with the TF/SC process. There are other BAF technology providers can be considered such as the US Filter (Biostyr®). Biosolids production using the BAF process is estimated to be about 30% higher than the TF/SC option; thus, the overall increase in sludge (primary and biological) for the BAF option requires about 20% more sludge handling capacity than the TF/SC process.

Discussions were held with Dégremont® regarding the sizing of a BAF plant using the Biofor® system. The BAF system consists of 30 modules complete with necessary clean effluent backwash water storage tank, dirty backwash water storage tank, aeration blowers for process air, backwash pumps and air scour blowers. The supplier has indicated that the optimal performance of BAF peaks at 80-85% removal for BOD. In order to achieve a design average effluent criteria of 20 mg/L for BOD, one option is to use CEP ahead of the BAF. Alternatively, a two-stage BAF treatment can be considered (80% removal in Stage 1 and additional 10~15% removal in Stage 2) to achieve the desired effluent quality. The preliminary capital cost estimates for BAF are based on a plant with one stage only.

4.1.2 Description of Options for Interim Upgrades

There are two objectives for interim upgrades:

- Meet the permit requirements for BOD and TSS concentrations
- Reduce effluent non-ammonia related acute toxicity

The selection of the preferred option for interim treatment depends on the objective. If the objective is to meet the permit requirements, interim options 1, 2 and 4 are applicable (see descriptions below). If the objective is to reduce effluent toxicity, options 2, 3 and 4 are applicable (see descriptions below). The selection of the interim upgrade option must also take into account the preferred options for build-out to secondary as well as the forecast of effluent quality reliability discussed in Section 3. Should TF/SC be selected as the preferred option for build-out to secondary, either Chemically Enhanced Primary (CEP) or Roughing Trickling Filter (RTF) or a combination of both would be a logical choice for interim upgrades. If BAF were selected for the upgrade of build-out to secondary, either CEP or BAF or a combination of both would be a logical choice for interim upgrades.

Interim Option 1 – RTF treating 25% ADWF

This option was added following the forecast of effluent quality carried out in Section 3 of this appendix 10. Based on the analysis in Section 3 this option will provide 99% TSS compliance, but will not achieve 99% BOD reliability up to the build-out to secondary expansion. In addition, the results of the small scale testing indicate that this option would not result in a significant reduction in toxicity.

Detailed description of partial biological treatment is included in the Interim Option 2 below.

Interim Option 2 - 50% RTF

Based on the results of the small scale testing, this option would result in a reduction in effluent toxicity. This option would also provide minimum 99% reliability until the build-out to secondary is carried out (before 2021).

The preliminary (screen and grit removal) and primary (primary sedimentation tank) treatment units will continue to treat the entire peak wet weather flow collected from the VSA. No additional primary sedimentation tanks will be provided. To prevent solids from entering the trickling filter, it would be necessary to add fine screens (6 mm) or to provide submerged effluent launders at the primary clarifiers.

The RTF process is designed to provide treatment of 50% average dry weather flow (ADWF) for a hydraulic capacity of 228 ML/d (50% x 456 ML/d = 228 ML/d), at 50% of the maximum month (MMF) loading of 81 t/d of BOD and 58 t/d of TSS after the primary treatment units. The RTF will be sized to treat 40.5 t/d and 29 t/d for BOD and TSS respectively.

Final clarifiers will be used to remove TSS and biological sludge generated from the RTF process. The portion of flow greater than 50% of the ADWF will bypass the secondary treatment units. The primary treated flow and the portion of the flow that receives biological treatment (a constant flow of 228 ML/d) would be combined and discharged directly to the outfall pumping station.

The primary sludge and the biological sludge are thickened in the gravity thickeners and dissolved air flotation (DAF) units, respectively. The thickened sludge from both streams will be blended and stabilized in the same anaerobic digesters to achieve volatile solids reduction and pathogen kill. The anaerobic digesters are designed to operate at mesophilic condition during the interim stage, with the design capability to be operated at thermophilic condition for efficiency improvement and future expansion (e.g. build-out to secondary). The digested biosolids will then be placed in the adjacent drying lagoon for dewatering and storage. The supernatant from the lagoon will be returned to the process for treatment. The dry solids will be stockpiled onsite or be transported to other land application sites. Because of the capacity limitation of the sludge lagoon (provision to fill one existing lagoon for solids handling facility expansion), mechanical dewatering will be provided to deal with additional sludge produced in the biological treatment.

Interim Option 3 – CEP only

This option was added following the forecast of effluent quality carried out in Section 3 of this Appendix. This option would provide minimum 99% reliability until the build-out to secondary treatment is carried out by 2021. Based on the results of the small scale testing, it appears that this option would result in a significant reduction in effluent toxicity.

The entire flow will receive preliminary and CEP treatment. However, CEP could be turned off when the effluent is highly diluted during long periods of wet weather flow. In order to handle the additional sludge produced by the CEP process, additional gravity thickeners and digesters are required.

Operating and maintenance costs for this option have been developed assuming a whole year round CEP operation. In order to reduce operating cost, CEP could be added only for flow equal or less than 1.5 times of the average annual flow (AAF) for example. Based on historical records and statistical analysis, this approach could reduce the days of CEP operation by 20 to 36 days (6 to 10 % of a calendar year). However, additional information can be considered into the decision such as on-line/instant COD monitoring and precipitation records in the VSA catchments. By establishing a correlation between effluent BOC/COD and monitoring the influent COD trends, it may be possible to determine when CEP could be turned on.

Interim Option 4 – CEP + 50% RTF with No Secondary Clarifiers

This option is a variation of Option 3 with the addition of chemically enhanced primary (CEP) treatment followed by partial biological treatment using RTF without secondary clarifiers. Chemicals (alum and polymer) are added prior to the primary sedimentation tanks in order to increase TSS and BOD removal efficiency.

This option would provide minimum 99% reliability until the build-out to secondary treatment is carried out by 2021. Based on the results of the small scale testing, this option appears that it would result in a significant improvement in effluent toxicity reduction.

The entire flow will receive preliminary and CEP treatment. However, CEP could be turned off when the effluent is highly diluted during long periods of wet weather flow. Following the CEP process, 50% of the ADWF (50% x 456 ML/d = 228 ML/d) will undergo the RTF process. The design loadings in the RTF process are for 50% of the maximum month loadings (MML), i.e. 28 t/d of BOD and 12 t/d of TSS. Because of the enhanced TSS removal provided by CEP, secondary clarifiers will not be provided after the RTF since the TSS levels in the effluent would meet the permit requirements. Projected TSS concentrations are included in Table 4.2 based on average annual flow and maximum month loading. However, the RTF will provide the required reduction of BOD in order to meet the permit requirements and/or effluent toxicity reduction.

The portion of the flow greater than 50% of the ADWF will bypass the secondary treatment units. The CEP treated flow and the portion of the flow that receives biological treatment would be combined and discharged directly to the outfall pumping station. The combined primary and chemical sludge will be collected in the primary sedimentation tanks and thickened in the gravity thickeners. Since there is no secondary clarifier to remove TSS and biological solids, no biological sludge will be produced and DAF will not be required.

4.1.3 Tank Size of Number of Units Needed.

The process design models developed in Appendix 3 and Appendix 4 were amended as follows:

- Design Case flows and loads were used
- PST performance as determined from the analysis of data (Section 3) replaced the standard values. No additional PST is required in the interim and build-out to secondary upgrades.
- Additional interim and build-out to secondary treatment options were added as described in Sections 4.1.1 and 4.1.2 above.

The modeling results are included in Appendix A. The unit process dimensions and number of process units required are summarized in Tables 4.1.

TABLE 4.1
TOTAL NUMBER OF UNIT PROCESS FOR PREFERRED OPTIONS AT IIWWTP

Unit Process	Interim 2021				Build out to Secondary 2036	
	Option 1 25% RTF	Option 2 50% RTF	Option 3 CEP only	Option 4 CEP + 50% RTF no SCL	Option 1 TF/SC	Option 2 BAF
Design Flow for Biological Treatment	114 ML/d	228 ML/d	0	228 ML/d	912 ML/d	912 ML/d
Additional Primary Sedimentation Tanks (PST)	0	0	0	0	0	0
Trickling Filter (TF) 44m dia. × 6 m high	1	2	0	2	6	0
Solids Contact (SC) 78 m × 8 m × 5 m	0	0	0	0	4	0
Biological Aerated Filters (BAF) 14 m ×10 m × 5 m	0	0	0	0	0	30
Secondary Clarifiers (SCL) 41 m dia. × 5 m	2	4	0	0	16	0
Gravity Thickeners (GT) 20 m dia. × 4 m	0	0	2	2	0	0
Dissolved Air Flotation (DAF) 21.5 m dia. × 3.5 m	1	2	0	0	3	3
Digesters 32m dia. × 10.6 m	1	2	3	3	4	4
Centrifuge 145 m^3/hr	2	3	3	3	4	4

4.1.4 Site Layout

Several layout options were examined for the location of the TF/SC secondary treatment plant and the additional solids handling facilities. The general layout options considered for the liquid stream for the TF/SC secondary treatment plant are shown on Figures 4.1, with colour-coded staging of four interim options and build-out to secondary with TF/SC process. The layout shown on Figure 4.1 was selected because it avoids most of the wetlands that occupies the south half of the site and also avoids the grit dump located in the east-central part of the site. By locating the liquid stream east of the existing primary sedimentation tanks, flow diversion to the secondary plant can be achieved with minimal head losses and pumping energy.

It is proposed to locate the solids handling facilities, which consist of DAF, digesters and centrifuge, to west of the existing plant in the area presently occupied by the north-east sludge lagoon. The proposed solids handling facilities would then be located near the existing digesters shown in Figure 4.1. The BAF upgrade options are shown in Figure 4.2, with interim and build-out to secondary staging.

4.1.5 Projected Effluent Quality at Maximum Month Load and Annual Average Flow

Projected effluent qualities (BOD and TSS concentrations) for each of the preferred option are shown in Table 4.2. For the interim upgrading options, the projected quality is based on the annual average flow condition (1.3 x ADWF) and maximum month loading (MML). Essentially, the biological process for the interim options would be operated at their maximum hydraulic design capacity all year around (228 ML/d), since the design AAF of 650 ML/d is significantly greater than 50% of ADWF (228 ML/d).

The projected effluent BOD and TSS concentrations for the build-out to secondary treatment are also summarized in Table 4.2. At the design flow condition (2 × ADWF), the effluent BOD and TSS concentrations are about 20 mg/L, respectively. Better effluent quality can be expected at the design AAF condition. The PWWF effluent BOD and TSS concentration are estimated about 30 mg/L and 25 mg/L respectively. Effluent toxicity reduction can be expected for all treatment options as a result of the provision of biological treatment.

FIGURE 4.1

LAYOUT OF BAF AT IIWWTP

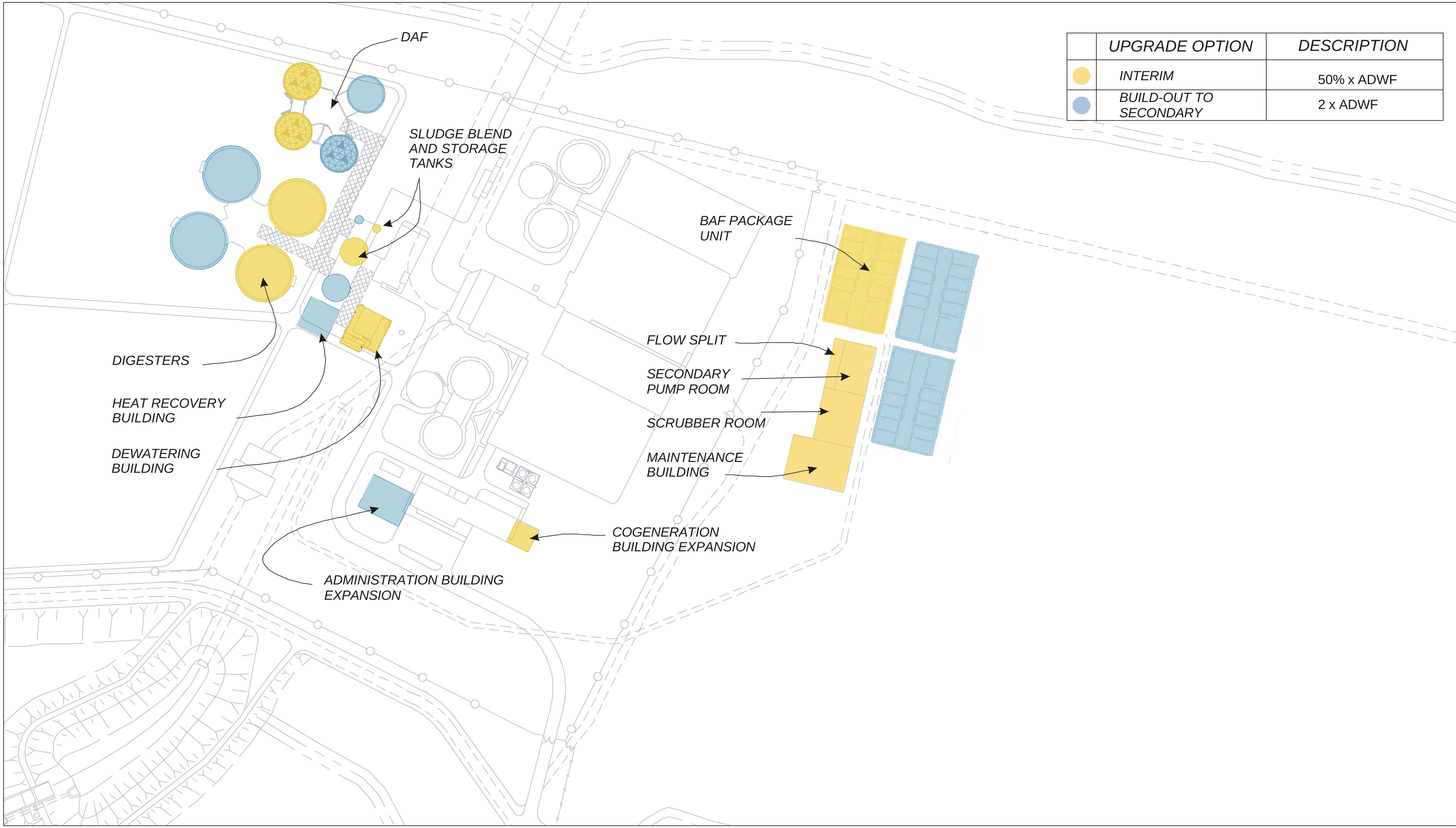


FIGURE 4.2

LAYOUT FOR TF/SC AT IIWWTP

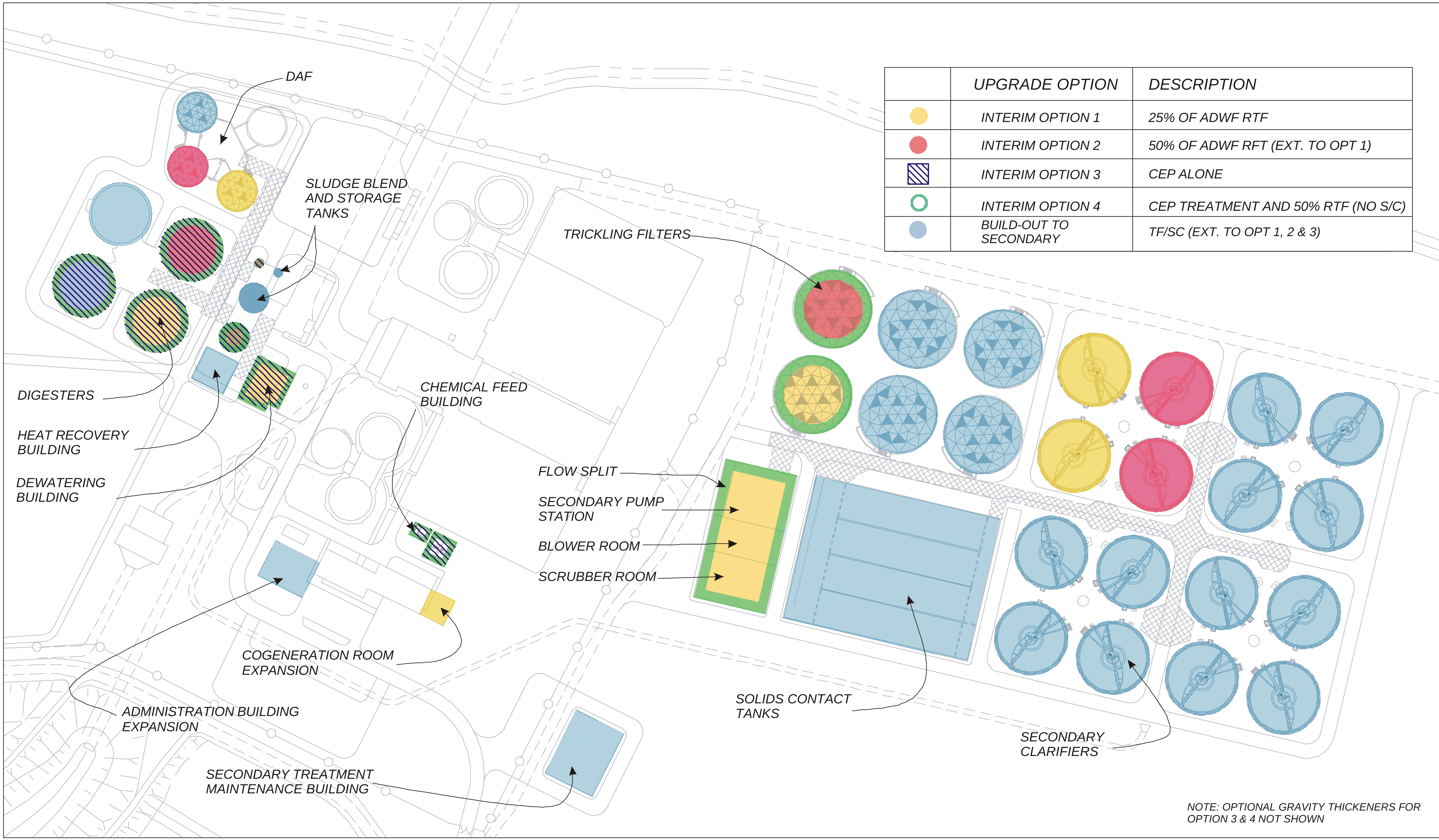


TABLE 4.2
IIWWTP PROJECTED EFFLUENT QUALITY AT MAXIMUM MONTH LOADING

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option A TF/SC	Option B BAF
Design Flow	AAF			2*ADWF		
Effluent BOD (mg/L)	114	93	94	64	20	20
Effluent SS (mg/L)	83	68	39	62	20	20

The solids handling facilities are sized on the basis of the maximum month loading. The estimated annual sludge production is based on average annual loading.

The projected sludge productions based on average annual loading for the preferred options are shown in Table 4.3 for the primary, chemical and biological sludges respectively. The sludge quantities are expressed in dry solids mass (tonnes/d) and are based on mesophilic anaerobic digestion. The estimated sludge volumes (m^3/d) at various sludge handling stages are also shown in Table 4.3, including the gravity thickener underflow, DAF supernatant, digested sludge and dewatered sludge. The increase in sludge production compared to the current averages of 920 m^3/d of digested sludge and 54 m^3/d dewatered sludge at 30% solids concentration are summarized in Table 4.4 for their dry weight and wet volumes. The solids content of the digested sludge and dewatered sludge concentrations are estimated about 2.7~3.6% and 27~35%, respectively.

With the provision that existing sludge stockpiling space will be used for plant expansion and one lagoon will be filled for solids handling facility expansion, the remaining lagoon capacity will not be sufficient to provide sufficient dewatering capacity. Mechanical or other types of dewatering will be required to dewater the digested sludge prior to hauling. The digested sludge and dewatered sludge concentrations are estimated about 3% and 27%, by using thermophilic anaerobic digestion and centrifuge respectively.

It takes six years to stabilize the sludge in the lagoons followed by an additional two years for drying. By providing mechanical sludge dewatering by 2012, the sludge contained in the lagoon would be removed by 2020.

Metal and nutrient concentration increases in CEP and biological sludge are expected with interim process upgrade from primary to partial secondary treatments. Such impact on biosolids reuse and recycling options should be further evaluated.

TABLE 4.3
IIWWTP ESTIMATED SLUDGE PRODUCTION (ANNUAL AVERAGE)

YEAR		Interim 2021				Build-out to Secondary 2036	
Option	Unit	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Raw Sludge/Biosolids							
Primary Sludge	t/d	48	48	-	-	50	50
CEP Sludge	t/d	0	0	91	91	0	0
Secondary Biosolids	t/d	12	23	0	0	42	56
Total Raw Sludge	t/d	60	72	91	91	92	105
Thickened Sludge							
Gravity Thickener	m^3/d	967	967	1,777	1,777	992	992
DAF Supernatant	m^3/d	334	669	0	0	1,202	1,591
Total Thickened Sludge	m^3/d	1,301	1,635	1,777	1,777	2,193	2,583
Digested Sludge	m^3/d	1,301	1,635	1,777	1,777	2,193	2,583
Dewatered Sludge	m^3/d	112	134	146	146	190	218

TABLE 4.4
IIWWTP INCREASE OF SLUDGE COMPARED TO CURRENT LEVEL (ANNUAL AVERAGE)

YEAR		Interim 2021				Build-out to Secondary 2036	
Option	Unit	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Raw Sludge	%	49	77	126	126	127	160
Thickened Sludge	%	41	78	93	93	138	181
Digested Sludge	%	41	78	93	93	138	181
Dewatered Sludge	%	107	147	169	169	250	303

4.1.6 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 4.5. Detailed breakdowns of the cost estimates are included in Appendix B. All capital cost estimates are expressed in November 2003 dollars.

For the interim period, it was initially assumed that the current solids handling method of lagoon storage followed by on-site stockpiling would continue. Following discussions at Workshop # 3, it is proposed to provide mechanical dewatering at the time of interim upgrades and the cost estimates have been revised to this effect.

TABLE 4.5
IIWWTP CAPITAL COST ESTIMATES

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
CAPITAL COSTS						
Site Improvements	$8,865,000	$21,775,000	$3,030,000	$13,046,000	$34,835,000	$21,034,000
Chemical Feed	$0	$0	$1,500,000	$1,500,000	$0	$0
Biological Aerated Filter	$0	$0	$0	$0	$0	$92,600,000
RTF/TF/SC	$8,395,000	$16,790,000	$0	$16,790,000	$50,370,000	$0
Solids Contact Tank	$0	$0	$0	$0	$10,541,000	$0
Secondary Clarifiers (SCL)	$5,649,600	$11,299,200	$0	$0	$45,196,800	$0
Gravity Thickeners	$0	$0	$2,772,000	$1,935,000	$0	$0
Sludge Blending Tank	$250,000	$500,000	$0	$0	$1,000,000	$1,000,000
DAF Thickeners	$7,695,600	$15,391,200	$0	$0	$23,086,800	$25,395,480
Digesters	$8,013,500	$16,027,000	$24,026,400	$24,026,400	$32,054,000	$35,259,400
Mechanical Dewatering	$7,000,000	$10,000,000	$10,000,000	$10,000,000	$25,800,000	$25,800,000
Site Works	$2,847,500	$4,002,500	$150,000	$3,135,000	$14,022,000	$7,982,000
Admin/Maint. Building	$0	$0	$0	$0	$4,000,000	$4,000,000
Control System	$1,855,000	$3,711,000	$1,593,000	$2,150,000	$6,610,000	$6,190,000
Expansion of Cogeneration	$1,500,000	$7,000,000	$9,900,000	$9,900,000	$11,700,000	$11,700,000
Odour Control	$500,000	$1,000,000	$0	$1,000,000	$3,000,000	$500,000
Existing Facility Upgrades	$11,000,000	$11,000,000	$11,000,000	$11,000,000	$69,150,000	$69,150,000
Sub-Total	$63,571,000	$118,496,000	$63,971,000	$94,482,000	$331,366,000	$300,611,000
Division 1 Cost	$1,368,000	$2,418,000	$1,524,000	$2,036,000	$7,413,000	$6,989,000
Engineering	$10,171,000	$18,959,000	$10,235,000	$15,117,000	$53,018,000	$48,098,000
Project Management/QA/QC	$2,543,000	$4,740,000	$2,559,000	$3,779,000	$13,255,000	$12,024,000
Contingency	$19,071,000	$35,549,000	$19,191,000	$28,345,000	$99,410,000	$90,183,000
Total Capital Costs	$96,724,000	$180,162,000	$97,480,000	$143,759,000	$504,462,000	$457,905,000

4.1.7 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs (November 2003 dollars) for interim upgrades at 2021 flow and for build-out to secondary at 2036 flows are shown in Table 4.6. In addition, the total O/M costs of the entire plant (existing primary and upgrade) and the costs of the upgrade plant only, are listed in Table 4.6, respectively. The existing primary plant has 57 staff. For the interim upgrade, it is estimated that the staff would increase to 65 persons. For the build-out to secondary it is estimated that the staff would increase to 80 persons. Maintenance costs are estimated at the existing maintenance cost of the existing cost ($2,776,000 in 2002) plus a fixed 0.8% of the capital cost.

The residual management costs are estimated based on a rate of $100/tonne for hauling, reuse (e.g. land application), and other fixed expenses, assuming that land application sites are available. The solids concentration is estimated to be about 30% using mechanical dewatering by centrifuge.

TABLE 4.6
IIWWTP OPERATING AND MAINTENANCE COST ESTIMATES

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
O&M COSTS						
Labour	$4,359,000	$4,695,000	$4,359,000	$4,695,000	$5,365,000	$5,365,000
Chemical Costs*	$0	$0	$7,847,000	$7,847,000	$450,000	$517,000
Residuals Management	$4,091,000	$4,888,000	$5,329,000	$5,329,000	$6,938,000	$7,970,000
Energy/Power	$1,136,000	$1,207,000	$1,188,000	$1,213,000	$2,458,000	$3,645,000
Repair/Maintenance	$3,550,000	$4,218,000	$2,930,000	$2,776,000	$6,812,000	$6,440,000
Administration and others	$1,671,000	$1,720,000	$1,671,000	$1,699,000	$1,784,000	$1,757,000
Total (O&M Costs)**	$14,805,000	$16,727,000	$23,323,000	$23,557,000	$23,806,000	$25,691,000
Total (O&M Costs)***	$5,714,000	$7,635,000	$14,231,000	$14,465,000	$14,714,000	$16,600,000

Notes
*: Polymer for dewatering is not included in Interim Option 1 and Option 2
**: Entire plant O/M costs including existing primary plant and upgrade
***: Upgrade O/M costs only (existing primary plant excluded)

4.1.8 Life Cycle Cost Analysis/Net Present Value Analysis

The life cycle costs (LCC) of each upgrade option are included in Table 4.7. The LCC are based on the following parameters:

- Discount rate: 6% p.a.
- Base date for costing: November 2003
- Evaluation period for interim: 2004 to 2020
- Evaluation period for build-out: 2004 to 2060
- Commissioning date for interim: 2009(see Section 3.1)
- Construction period for interim: 2008-2009 (1/2 of capital each year)
- Construction period for build-out: 2018-2020 (1/3 of capital each year)

The cash flow details of the capital and O/M costs are included in Appendix D. The O/M costs considered in the LCC analysis include the upgrade plant only, excluding the existing primary plant operation. The LCC analysis differs from the capital works program in the timing assumed for expenditures. This difference results from the uncertainties associated with the needs for upgrading.

The capital program identifies the capital need for upgrading the plants between the present and the date when build-out to secondary is to take place. The timing of the interim upgrades is dependent on a number of factors including the effluent BOD and TSS reliability against the permit requirement and the need to improve the toxicity test results of 100% effluent samples and monitoring of the receiving environment.

The LCC analysis allows the comparison of cost of meeting effluent quality needs using different treatment processes to be compared on a common basis. In the LCC analysis, the cost of each option is independent and stand-alone. The cost of interim treatment cannot be added to the cost of build-out. This is because it would be necessary to deduct the capital work of the interim works from the build-out work to arrive at a correct result. As indicated above, the LCC analysis of the interim options has been assumed that the upgrades will be constructed by 2011. No operating costs have been included for the build-out options between 2004 and 2021. This is because the costs are dependent on the interim option chosen.

TABLE 4.7
IIWWTP TREATMENT OPTION LIFE CYCLE COST

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Discounted Total O&M Cost	$31,770,000	$42,451,000	$79,124,000	$80,425,000	$82,217,000	$92,756,000
Discounted Capital Costs	$70,233,000	$130,818,000	$70,782,000	$104,385,000	$198,805,000	$180,457,000
Total Capital and O & M Costs at Present Value	$102,002,000	$173,268,000	$149,905,000	$184,810,000	$281,022,000	$273,212,000

4.1.9 Proposed Interim Upgrade Schedule

The scheduling and need for interim upgrades depends on the objectives to comply with the permit requirements (i.e. 100 mg/L of TSS and 130 mg/L of BOD), or to achieve effluent toxicity reduction.

Effluent Quality Compliance

The forecast of effluent quality included in Section 3.1 indicates that if the loads increase in the near term, the current PST treatment will not meet a 99% effluent quality reliability objective without immediate treatment upgrade. With Interim Option 1 (25% biological treatment), 99% reliability for BOD would be met until about 2015. With any of the other interim options, 99% reliability for BOD would be met until the build-out to secondary is completed by 2021.

Effluent Toxicity Reduction

Improvement of effluent toxicity reduction can be achieved with minimum 50% ADWF of biological treatment. The results of small-scale testing also suggested a significant effluent toxicity reduction is possible with CEP only.

4.1.10 Approaches to Interim Treatment Upgrade

Several possible approaches to upgrade the interim treatment are outlined in Table 4.8. Option 2, 3, and 4 could achieve a minimum 99% reliability to meet effluent objective and significant effluent toxicity reduction (small-scale testing showed approximate 60% improvement of toxicity reduction) before 2021. Option 1 could meet the effluent reliability of 99% until 2015, however, no significant effluent toxicity reduction could be expected based on the small-scale testing results.

Due to site preparation requirements at IIWWTP for interim upgrade, proposed schedule of various upgrade options are summarized in Table 4.9, which include emptying lagoons, removing stockpiles, pre-loading and design/construction.

TABLE 4.8
APPROACHES TO INTERIM UPGRADES AT IIWWTP

Interim Option	Effluent BOD & TSS Reliability above 99%	Effluent Toxicity Reduction	Remarks
TF/SC for 25% of ADWF	• To 2021 for TSS • To 2015 for BOD	No	• 1/8 of build-out to secondary plant capacity • Similar capital cost to option 3 ($96.8M vs. $97.5M) • Lowest LCC.
TF/SC for 50% of ADWF	• To 2021 for TSS and BOD	60% (based on small scale testing)	• 1/4 of build-out to secondary plant capacity • Highest capital cost ($180.2M)
CEP only	• To 2021 for TSS and BOD	60% (based on small scale testing)	• Can be operated intermittently • Potentially generates largest quantity of sludge • Similar capital cost to Option 1 ($97.5M vs. $96.8M), however higher LCC than Option 1. • Allows postponement for the selection of biological process
CEP + TF/SC for 50% of ADWF and no secondary clarifier	• To 2021 for TSS and BOD	60% (based on small scale testing)	• Provides flexibility • Potentially generates largest quantities of sludge • Less expensive than Option 2 ($143.8M vs. $180.2M) capital cost, however, highest LCC. • No secondary clarifiers

TABLE 4.9
PROPOSED SITE PREPARATION AND DESIGN/CONSTRUCTION SCHEDULE FOR IIWWTP

Option and Stage Work	Interim Design Flow - Year 2021				Build-out to Secondary Design Flow - Year 2036	
	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Decision Review	2005	2005	2005	2005	2015	2015
Design and Tender	2006~2007	2006~2007	2006~2007	2006~2007	2016~2017	2016~2017
Construction	2008~2009	2008~2009	2008~2009	2008~2009	2018~2020	2018~2020
Empty Lagoon[1)]	2006	2006	2006	2006	-	-
Remove Stockpiles	2006[2)]	2006[2)]	-	2006[2)]	2026[5)]	2026[5)]
Preloading	2007[3)]	2007[3)]	2007[3)]	2007[3)]	2016~2017[6)]	2016~2017[6)]
Sludge Pre-treatment Pilot Trial[4)]	2005	2005	-	2005	-	-

Notes:
[1)]: Empty #1 or #4 lagoon and use the remaining lagoon during interim
[2)]: Remove stockpile in west half of site of proposed expansion
[3)]: Preloading of part of lagoon cell and west half of site for expansion
[4)]: e.g. ultrasonic pre-treatment
[5)]: Remove sludge and grit stockpile in east half of site for expansion
[6)]: Preloading of east half of site for expansion

4.2 LIONS GATE

4.2.1 Description of Options for Build-out to Secondary

The selection of the final recommended process is addressed in detail in Section 10 of Appendix 3 and Section 9 of Appendix 4. The selected process is BAF based on the small footprint, which allows expansion beyond the required capacity forecast for 2046.

The headworks of the plant have not been addressed in detail as part of this study. However it is recognized that the following upgrading is needed:

- The capacity of the inlet pumping station is 298 ML/d with no redundancy .
- Screening system capacity increased to match the inlet pump station capacity with adequate redundancy.
- Grit removal upgrade, if justified on business grounds, to improve the capture and match inlet pump capacity

The existing capacity of 298 ML/d of the inlet pumping station and of the headworks corresponds to about 2.6 times the projected ADWF for the year 2046. If the capacity of the headworks is not to be increased a wet weather management program will be required to address any increase in wet weather flow beyond the capacity of the inlet pumping station. Because of the long time frame for the flow projections, it is assumed that corrective measures will be undertaken to reduce inflow and infiltration. The lack of redundancy will also need to be addressed.

The means by which these upgrades will be achieved is not determined. Consideration should be given to the use of Archimedes screw pumps to raise the raw sewage to ground level where after screening and grit removal facilities could be installed. A layout for vortex grit removal, which could be incorporated in the present configuration, is shown on the drawings (Volume 5, Iona Island Wastewater Treatment Plant, Interim and Build-out to Secondary Stage Preliminary Design Drawings).

Biological Aerated Filters

The existing PSTs will be retained without upgrading. Following these, a flow of twice the average dry weather flow (ADWF) of 111 ML/d = 222ML/d is pumped into a BAF system. Flows in excess of 2 x ADWF discharge from the PSTs to the chlorine contact tank and are then blended with the secondary effluent.

The BAF system consists of 10 modules complete with necessary clean effluent backwash water storage tank, dirty backwash water storage tank, aeration blowers for process air, backwash pumps and air scour blowers. The design load on the BAF plant maximum month loading (MML) is 20.4 t/d BOD and 8.8 t/d of TSS. The BOD load limits the treatment capacity of the plant. Back washing is triggered by the build-up of biomass in the filter resulting in increased head losses. The frequency with which back washing can be carried out determines the capacity.

Backwash water from the BAF is treated in a dissolved air flotation (DAF) system with the effluent (subnatant) discharged to the BAF influent stream.

Primary sludge from the PSTs is thickened in gravity thickeners. Additional thickener capacity is required.

The sludge is treated in thermophilic anaerobic digesters and de-watered in the present centrifuge plant which will be upgraded.

Centrate is discharged to the plant influent stream.

Effluent from the BAF can be disinfected by the recently upgraded hypochlorite chlorination and dechlorination system which is operated from April to October. The PST effluent would be chlorinated and discharged to the chlorine contact tanks, which would be retained.

4.2.2 Description of Options for Interim Treatment

The selection of unit operations is restricted to those, which are compatible with the Build-out to Secondary.

In order to ensure BOD effluent compliance until 2031 either CEP treatment or partial biological treatment is required. The use of combined CEP and biological treatment offers the opportunity for a further improvement in the effluent quality over partial biological treatment. The treatment of centrate may extend the date on which interim treatment is required. In order to allow assessment of the merits of various options the following options were assessed:

- Option 1 - CEP
- Option 2A - 50% ADWF BAF
- Option 2B - CEP + 50% ADWF BAF

Upgrading of the headworks of the plant is required as described in 4.2.1 above.

Option 1 – CEP

The existing PSTs are retained without increase in area. The CEP process is sized to provide 70 mg/l of Alum upstream of the PSTs. The required upgrading of the solids handling systems is assessed. This indicates a requirement for an increase in the sludge thickener capacity and in the digester capacity. Centrate discharges to the plant influent stream. Disinfection using chlorine is continued.

Option 2A – 50% ADWF BAF

The existing PSTs are retained without increase in area. Flow downstream of the PSTs is pumped to a BAF with a capacity of 50% of ADWF = 50% x 104 ML/d = 52 ML/d. The BAF is sized to treat the load associated with a plant of 50% ADWF capacity. As indicated in Appendix 3 Figure 9.22, the BOD load is 50% of the total load on the plant. In this case the loads are 9.0 t/d BOD and 5.5 t/d of TSS MML. Supporting unit processes required are similar to those required for Build-out to Secondary. Disinfection using chlorine is continued.

Option 2B – CEP plus 50 % ADWF BAF

The plant is configured as for Option 2A with the addition of CEP dosing facilities. The capacity of the BAF has been retained at the same level as for Option 2A so that the implications of partial CEP treatment can be assessed.

4.2.3 Requirements for Unit Process Upgrading

The process models used for studies reported on in Appendices 3 and 4 were amended as follows:

- Design Case flows and loads were used
- PST performance as determined from analysis of data replaced standard values
- Digester performance as determined from analysis of plant data replaced standard values

There are two paths possible for upgrading through Interim Treatment to Build-out to Secondary. These are (1) using CEP until 2031 followed by Build-out to Secondary, or (2) constructing 50% ADWF biological treatment as an interim stage. The results of the modeling are presented showing the development of these paths.

TABLE 4.10
TOTAL NUMBER OF PROCESS UNITS FOR PREFERRED OPTIONS AT LGWWTP

	Interim Treatment Using CEP Alone		Interim Treatment Using 50% ADWF BAF		
	Interim	Build-out	Interim		Build-out
	Option 1 CEP	Option3 2xADWF BAF	Option 2A 50% ADWF BAF	Option 2B CEP 50% ADWF BAF	Option 3 2xADWF BAF
Inlet PS Upgrade	-	yes	-	-	yes
Screening Upgrade	-	Yes	-	-	yes
Grit Removal Upgrade 8.5 m dia	0	2	0	0	2
Chemical Dosing	2	2	2	2	2
PSTs	0	0	0	0	0
BAF	0	10	6	6	10
Gravity Thickener 13.7 m dia	1	1	1	1	1
DAF* 15.0 m dia.	0	2	1	1	2
Anaerobic Digesters 22 m dia., 10.1 m depth	2	2	1	2	2
Centrifuge	0	1	0	0	1

Note: Existing plant process units not included above.
*: Redundancy not a concern – could be thickened in PSTs.

To operate the additional plant as set out above, 2, 3, and 3 additional operators are required in the interim upgrade Option 1, Option 2A and 2B, respectively. Four (4) additional operators are required for the build-out to secondary upgrade (Options 2 and 3).

4.2.4 Rationale for Site Layout

The principle driver is to allow expansion of treatment capacity beyond the 2046 design capacity. Opportunities for increase in capacity of all units, which are crucial for the ability to expand the plant, are therefore shown in addition to those required for the expansion to 2046 capacity. Because of the space requirements to access individual units, the minimum number of units consistent with redundancy requirements has been chosen. While this makes staging somewhat more expensive initially, it allows more capacity to be accommodated on the site. Final design will need to examine carefully strategies for allowing maximum build-out. These strategies could include the use of rectangular tankage where possible, over sizing units to allow for future growth and the use of multi-level structures. Provision has been made for upgrading the head works adjacent to the existing facility, extended PSTs, and additional units for vortex degritters, BAF, DAF and anaerobic digesters. The gravity thickeners have spare capacity. Provision has been made for UV disinfection of secondary effluent

Two interim strategies have been shown. These illustrate the CEP and biological options both leading to the same secondary treatment configuration. The proposed layouts are shown on Figures 4.3 and 4.4 respectively.

FIGURE 4.3
LGWWTP PROPOSED PLANT LAYOUT AND STAGING
CHEMICALLY ENHANCED TREATMENT TIL 2031

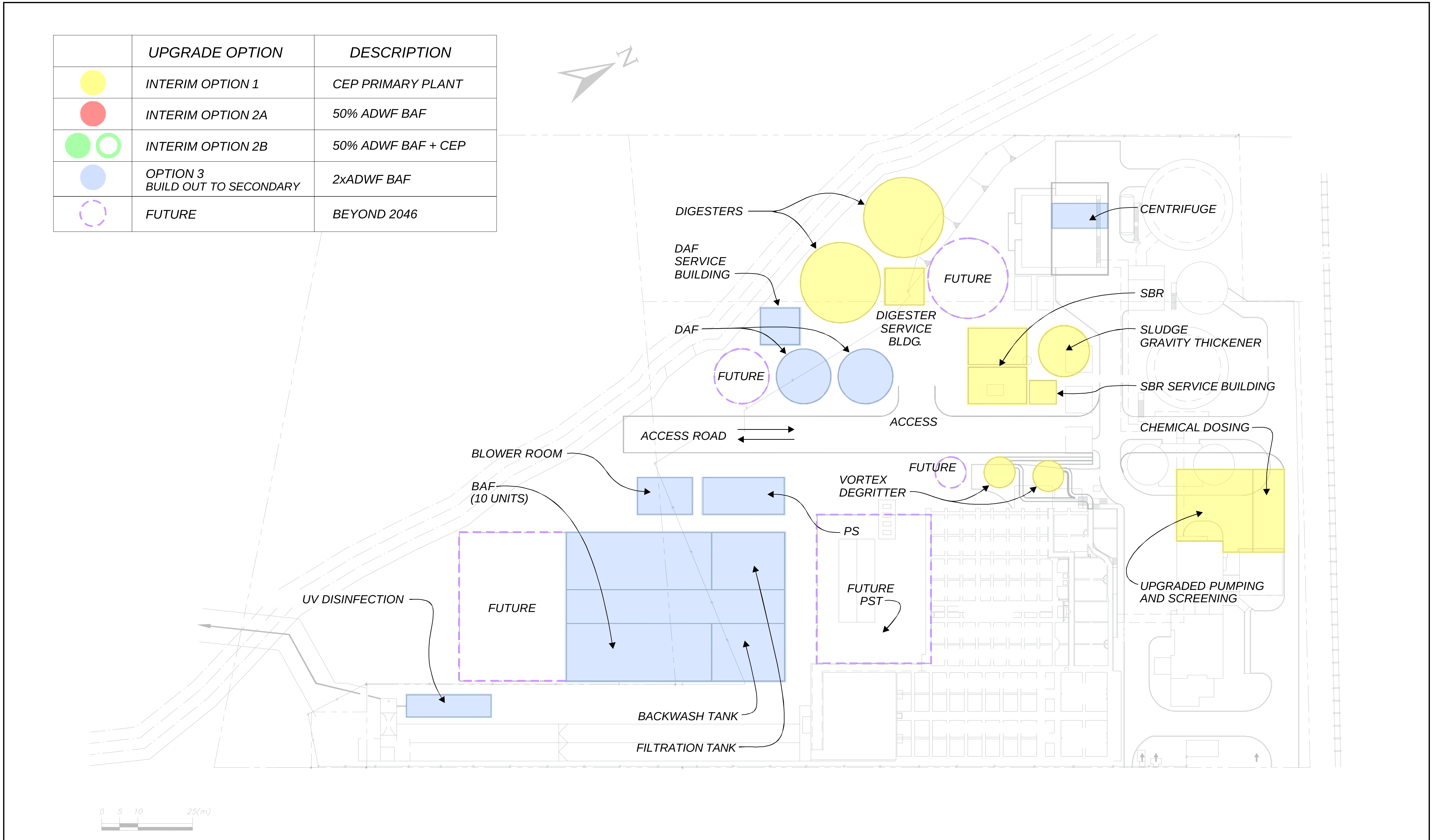
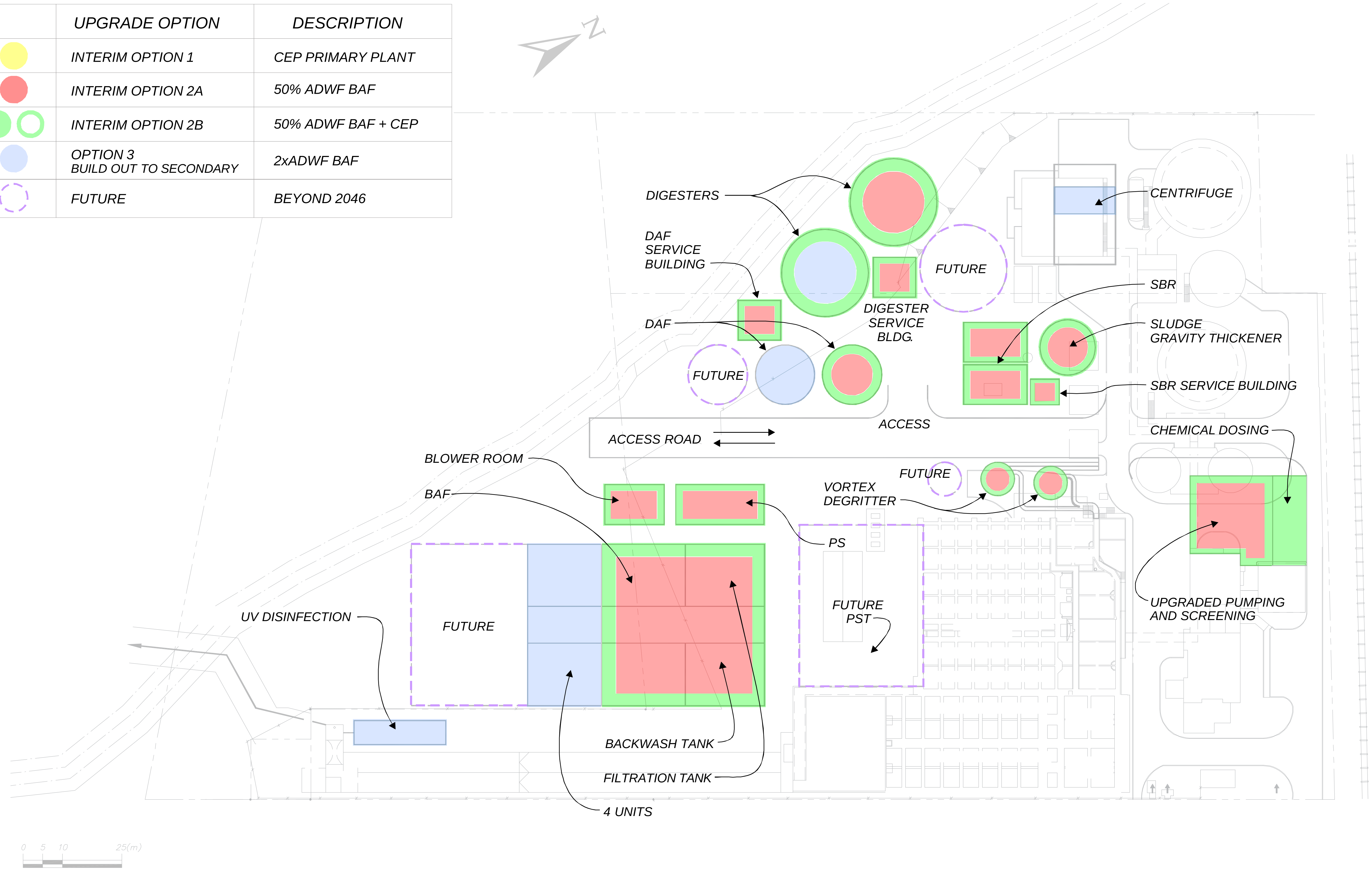


FIGURE 4.4
LGWWTP PROPOSED PLANT LAYOUT AND STAGING
PARTIAL BIOLOGICAL TREATMENT TIL 2031

	UPGRADE OPTION	DESCRIPTION
(yellow circle)	INTERIM OPTION 1	CEP PRIMARY PLANT
(red circle)	INTERIM OPTION 2A	50% ADWF BAF
(green circle / green ring)	INTERIM OPTION 2B	50% ADWF BAF + CEP
(blue circle)	OPTION 3 BUILD OUT TO SECONDARY	2xADWF BAF
(dashed purple circle)	FUTURE	BEYOND 2046



4.2.5 Projected Effluent Quality

Projected effluent qualities (BOD and TSS concentrations) for each of the preferred option are shown in Table 4.11. For the interim upgrading options, the projected quality is based on the annual average flow and maximum month load. Essentially, the biological process for the interim options would be operated at the maximum hydraulic design capacity all year around, since the design AAF over the period to 2031 of 108 to 125 ML/d is significantly larger than 50% of ADWF (52 ML/d).

The projected effluent BOD and TSS concentrations for the build-out to secondary treatment are also summarized in Table 4.11. At the design flow condition (2 x ADWF), the annual average effluent BOD and TSS concentrations are expected to be 20mg/L. Better effluent quality can be expected at the design AAF condition. Effluent toxicity reduction can be expected for all treatment options.

TABLE 4.11
LGWWTP PROJECTED EFFLUENT QUALITY AT ANNUAL AVERAGE FLOW AND MAXIMUM MONTH LOAD

	INTERIM			BUILD-OUT
	2031	2031	2031	2046
Option	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Design AAF/2*ADWF				
BOD (mg/L)	90	80	53	20
SS (mg/L)	45	52	31	20

4.2.6 Sludge Production Projections

The solids handling facilities are sized on the basis of the maximum month loading. The estimated annual sludge production is based on average annual loading.

The projected sludge production based on average annual loading for the preferred options are shown in Table 4.12 for the primary, chemical and biological sludge. The sludge quantities are expressed in dry solids. The estimated sludge volumes at various sludge handling stages are also shown in Table 4.12, including the gravity thickener underflow, DAF float, digested sludge and dewatered sludge. The increase in sludge production compared to current averages of 197 m^3/d of raw/digested sludge and 12 m^3/d dewatered sludge at 35% solids concentration is summarized in Table 4.13 for their dry weight and wet volumes. The solids content of the digested sludge and dewatered sludge concentrations are estimated to be 2.1~2.5% and 27~35%, respectively.

TABLE 4.12
LGWWTP ESTIMATED ANNUAL AVERAGE SLUDGE PRODUCTION

YEAR		INTERIM			BUILD-OUT
Option	Unit	2031	2031	2031	2046
		CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Raw Sludge/Biosolids					
Primary Sludge	T/d	16	12	-	13
CEP Sludge	T/d	3	0	19	0
Secondary Biosolids	T/d	0	12	7	12
Total Raw Sludge	T/d	19	24	25	26
Thickened Sludge					
Gravity Thickener (5%)	m^3/d	372	239	372	269
DAF (3.5%)	m^3/d	0	333	198	350
Total Thickened Sludge	m^3/d	372	572	569	619
Digested Sludge	m^3/d	372	572	569	619
Dewatered Sludge	m^3/d	22	43	45	57

TABLE 4.13
LGWWTP INCREASE OF SLUDGE COMPARED TO CURRENT LEVEL

YEAR		INTERIM			BUILD-OUT
Option	Unit	2031	2031	2031	2046
		CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Raw Sludge	%	89	140	159	161
Thickened Sludge	%	89	190	189	214
Digested Sludge	%	89	190	189	214
Dewatered Sludge	%	89	267	287	389

4.2.7 Capital Cost Estimates

The estimated capital costs of each upgrade option are shown in Table 4.14. Detailed breakdowns of the cost estimates are included in Appendix B. All capital cost estimates are expressed in November 2003 dollars.

The capital costs include the following:

- Seismic upgrading of the existing site by providing ground improvement in a berm along the shore.
- Soil anchors to reduce the probability of flotation of existing structures.
- Division 1 (2.5%), engineering (16%), project management/quality control (4%), contingency (30%) and GST (0%).

The following capital cost allowances have been included. It should be noted that these are allowances not estimates since the need for these upgrades has not been established.

- Upgrading of the inlet pump station.
- Upgrading of the inlet screens.
- Upgrading of grit removal facilities.

4.2.8 Operating and Maintenance Cost Estimates

The estimated operating and maintenance costs (November 2003 dollars) for interim upgrades at 2031 flow and for build-out to secondary at 2046 flows are shown in Table 4.15. The existing primary plant has a staff of 12. For the interim upgrade it is estimated that the staff would increase to 14 persons for Option 1 and to 15 persons for Options 2A and 2B. For the build-out to secondary it is estimated that the staff would increase to 16 persons. Maintenance costs are estimated at the existing cost plus a fixed 0.8% of the capital cost.

The residual management costs are estimated based on a rate of $100/tonne (wet solids) for hauling, reuse (e.g. land application), and other fixed expenses, assuming that land application sites are available.

TABLE 4.14
LGWWTP CAPITAL COST ESTIMATES

YEAR	INTERIM			BUILD-OUT
Option	2031	2031	2031	2046
	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
CAPITAL COSTS				
Site Improvements	$4,056,768	$4,056,768	$4,056,768	$4,466,127
Chemical Dosing	$500,000	$500,000	$500,000	$0
Primary Clarifiers	$0	$0	$0	$0
Bioreactor	$0	$14,524,084	$14,524,084	$23,649,647
Gravity Thickeners	$663,351	$663,351	$663,351	$663,351
DAF Thickeners	$0	$3,692,278	$3,692,278	$7,458,941
Digesters	$8,885,222	$4,442,611	$8,885,222	$8,885,222
Mechanical Dewatering	$0	$0	$0	$1,254,277
UV	-	-	-	$2,220,000
Odour Control System	$500,000	$500,000	$500,000	$500,000
Site Works	$362,920	$1,614,381	$1,614,381	$3,667,800
Admin/Maint. Building	$1,300,000	$1,300,000	$1,300,000	$2,000,000
Control System	$421,943	$972,893	$1,150,597	$1,785,258
Electrical Substation (allow)	$65,000	$85,000	$75,000	$115,000
Existing Facility Upgrades	$200,000	$200,000	$200,000	$14,200,000
Sub - Total	$16,955,205	$32,551,366	$37,161,682	$70,865,623
Division 1 Cost	$322,461	$712,365	$827,623	$1,659,987
Engineering	$2,712,833	$5,208,219	$5,945,869	$11,338,500
Project Management/QA/QC	$678,208	$1,302,055	$1,486,467	$2,834,625
Contingency	$5,086,561	$9,765,410	$11,148,505	$21,259,687
Total Capital Cost	$25,756,000	$49,540,000	$56,571,000	$107,959,000

TABLE 4.15
LGWWTP OPERATING AND MAINTENANCE COST ESTIMATES

YEAR	INTERIM			BUILD-OUT
Option	2031	2031	2031	2046
	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
O&M COSTS				
Labour	$1,626,268	$1,742,430	$1,742,430	$1,858,592
Chemical Costs	$1,604,770	$179,021	$1,653,213	$140,420
Biolite Replenishment	$0	$27,000	$20,202	$50,171
Residuals Management	$803,957	$1,562,839	$1,647,160	$2,082,794
Energy	$440,880	$564,978	$564,978	$862,519
Repair/Maintenance	$1,536,117	$1,726,389	$1,782,637	$2,193,741
Adminstration and others	$808,525	$745,661	$713,167	$721,242
Land and Building Lease	$331,839	$331,839	$331,839	$331,839
Total (O&M Costs)*	$7,153,000	$6,881,000	$8,456,000	$8,242,000
Total (O&M Costs)**	$3,005,000	$2,733,000	$4,308,000	$4,094,000

Notes

*: Entire plant O/M costs including existing primary plant and upgrade

**: Upgrade O/M costs only (existing primary plant excluded)

4.2.9 Life Cycle Cost Analysis and Net Present Value Analysis

The life cycle costs (LCC) of each option are included in Table 4.16. The O/M cost prior to interim is omitted. The LCCs are based on the following parameters:

- Discount rate: 6% p.a.
- Base date for costing: November 2003
- Evaluation period for interim: 2004 to 2016
- Evaluation period for build-out: 2004 to 2060
- Commissioning date for interim: 2017
- Commissioning date for build-out: 2031
- Construction period for interim: 2015-2016(1/2 of capital each year)
- Construction period for build-out: 2028-2030 (1/3 of capital each year)
- O/M cost basis: Upgrade net cost, present costs excluded

TABLE 4.16
LGWWTP LIFE CYCLE COST ESTIMATE

STAGE	INTERIM			BUILD-OUT
YEAR	2031	2031	2031	2046
OPTION	CEP ONLY	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Discounted O&M Cost	$13,095,343	$11,910,007	$18,773,623	$11,685,852
Discounted Capital Costs	$12,437,680	$23,923,074	$27,318,373	$23,757,336
Total Discounted Capital and O & M Costs at present value	$25,534,000	$35,834,000	$46,092,000	$35,444,000

The cash flow details of the capital and O/M costs are included in Appendix D.

The LCC analysis allows the comparison of the cost of meeting effluent quality needs using different treatment process to be compared on a common basis. In the LCC analysis, the cost of each option is independent and stand-alone. The cost of interim treatment cannot be added to the cost of build-out because it is necessary to deduct the capital cost of the interim works from the cost of the build out to arrive at the correct answer. As indicated above, in the LCC analysis of the interim options it has been assumed that the upgrades are constructed by 2017. No operating costs have been included for the interim options between 2004 and 2017 or for the build-out option between 2004 and 2031.

The Option 2B allows for full CEP followed by BAF treatment. An opportunity will probably exist to reduce the chemical dosage, which will reduce the operating costs.

4.2.10 <u>Proposed Schedule</u>

The scheduling and need for interim upgrades depends on the objectives to comply with the permit requirements (i.e. 130 mg/L for TSS and BOD) or to achieve effluent toxicity reduction.

Effluent Quality Compliance

The forecast of effluent quality included in Section 3.2 indicated that the loads increase as projected in the design case, the current PST treatment will not meet the 99% effluent quality reliability for BOD in 2017. With all of the interim options, 99% reliability for BOD would be achieved until the build-out to secondary is completed by 2031.

Effluent Toxicity Reduction

Effluent toxicity reduction can be achieved with biological treatment. The results of small-scale testing also suggested a significant effluent toxicity reduction is possible with CEP only.

4.2.11 Approach to Interim Treatment

The approaches to improve future compliance with the permit limits on BOD are:

- reduction in centrate load
- CEP
- partial biological treatment

In order to clarify the strategy the following actions are suggested:

1. Assess the impact of centrate load on effluent quality.
2. Assess options for treatment of centrate. One possibility is the use of a small BAF to provide BOD and possibly ammonia removal. This would have the advantage of allowing experience of BAF to be obtained before committing to a full-scale secondary plant.
3. Assess effectiveness of CEP at full-scale and the extent to which dosing would be required over time.
4. Assess the impact of CEP on sludge quality.
5. Assess the capacity of partial biological treatment required to achieve desired improvement in effluent quality.
6. Assess the combined use of the above options to determine the most effective combination.

4.2.12 Comment

Investment in demand management and I&I control will reduce the need to upgrade inlet pumping, screening and grit removal capacity. However the resulting increase in BOD concentration of the influent will advance the need for interim treatment.

APPENDIX A: PROCESS DESIGN SUMMARY

IIWWTP Process Design: Design Case Scenario

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Average Dry Weather Flow (ML/d), ADWF	456	456	456	456	456	456
Average Annual Flow (ML/d), AAF	593	593	593	593	573	573
Peak Wet Weather Flow (ML/d), PWWF	1,530	1,530	1,530	1,530	1,530	1,530
Maximum Month BOD Loading (t/d), MM BOD	124	124	124	124	127	127
Maximum Month TSS Loading (t/d), MM TSS	116	116	116	116	119	119
Primary Clarifier						
Average Annual Flow (ML/d)	593	593	593	593	573	573
Peak Wet Weather Flow (ML/d)	1,530	1,530	1,530	1,530	1,530	1,530
Overflow Rate ($m^3/m^2/d$) - AAF	55	55	55	55	55	55
Overflow Rate ($m^3/m^2/d$) - PWWF	130	130	130	130	130	130
Surface Area (m^2) - AAF	10,782	10,782	10,782	10,782	10,418	10,418
Surface Area (m^2) - PWWF	11,769	11,769	11,769	11,769	11,769	11,769
Depth (m)	2.74	2.74	2.74	2.74	2.74	2.74
Volume (m^3) - AAF	29,542	29,542	29,542	29,542	28,546	28,546
Volume (m^3) - PWWF	32,248	32,248	32,248	32,248	32,248	32,248
Raw Influent BOD Loading (t/d)	124	124	124	124	127	127
Raw Influent TSS Loading (t/d)	116	116	116	116	119	119
Return Influent BOD Loading (t/d)						
Return Influent TSS Loading (t/d)						
Total Influent BOD Loading (t/d)	124	124	124	124	127	127
Total Influent TSS Loading (t/d)	116	116	116	116	119	119
Total Influent BOD Conc.@ AAF (mg/L)	209	209	209	209	222	222
Total Influent TSS Conc.@ AAF (mg/L)	196	196	196	196	208	208
Design PC BOD removal (%)	35%	35%	55%	55%	35%	35%
Design PC TSS removal (%)	50%	50%	80%	80%	50%	50%
PC Effluent BOD Loading (t/d)	81	81	56	56	83	83
PC Effluent TSS Loading (t/d)	58	58	23	23	60	60
PC Effluent BOD Conc. @ AAF (mg/L)	136	136	94	94	144	144
PC Effluent TSS Conc. @ AAF (mg/L)	98	98	39	39	104	104
Chemical Usage	N/A	N/A			N/A	N/A
Alum Dosage (mg/L)			70	70		
Polymer Dosage (mg/L)			0.5	0.5		
Alum Volume - AAF (m^3/d)			63.2	63.2		
Polymer Volume - AAF (m^3/d)			0.7	0.7		
Alum Volume - PWWF (m^3/d)			163.1	163.1		
Polymer Volume - PWWF(m^3/d)			1.9	1.9		
Biological Treatment						
Treating % of ADWF	25%	50%		50%	200%	200%
Design Flow (ML/d)	114	228		228	912	912
Treating % of MM BOD/TSS loading	25%	50%		50%	100%	100%
Design BOD Loading (t/d)	20.15	40.3		27.9	82.55	82.55
BAF	N/A	N/A	N/A	N/A	N/A	
Sludge Yield						0.94
Sludge Age (days)						2
Design BOD Loading ($kg/m^3/d$)						5.50
Design TSS Loading ($kg/m^3/d$)						4.60
Design Hydraulic Loading $m^3/m^2/d$-average						150
Design Hydraulic Loading $m^3/m^2/d$-peak						240
Backwash flow Ml/d (10% of flow treated)						57
Reactor Area (m^2) -TSS						3496
Reactor Area (m^2) - BOD						4057
Reactor Area (m^2) - average flow						4202
Reactor Area (m^2) - peak flow						4039
Depth of media (m)						3.7
Volume Required (m^3) - based on Max Reactor Area						21010
Oxygen Requirement (kg O_2/kg BOD_5)						1.6
Actual Oxygen Transfer Rate AOTR (t/d)						102.9
SOTR (t/d O2)						226.4
Air requirement (sCFM)						67911
Design Effluent BOD Concentration (mg/L)						20
Design Effluent TSS Concentration (mg/L)						20

IIWWTP Process Design: Design Case Scenario (Cont'd)

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
RTF - Sidestream U/S of PST (via Fine screen)			N/A		N/A	
Design Fine Screen BOD Removal (%)						
Design Fine Screen TSS Removal (%)						
BOD Loading after Fine screen (t/d)	20	40		28		
TSS Loading after Fine screen (t/d)	29	58		12		
Design Trickling Filter Loading (kg BOD/m^3/d)	3.5	3.5		3.5		
Volume of Trickling Filter (m^3) - organic load	5757	11514		7971		
Hydraulic Loading rate (m^3/m^2.d) Min = 45	100.0	100.0		100.0		
Area of Trickling Filter (m^2) by hydraulic loading	1140	2280		2280		
Depth of Tower (m)	5.1	5.1		3.5		
Design sBOD removal (%)	61%	61%		61%		
Sludge Age (days)	3	3		3		
Sludge Yield (kg TSS/kg BOD)	0.81	0.81		0.81		
sBOD (%) of TBOD	35%	35%		35%		
Biodegradable TSS (%)	80%	80%		80%		
Effluent BOD (t/d)	2.8	5.6		3.9		
Effluent SBOD (mg/L)	24.4	24.4		16.9		
Effluent TSS (t/d)	16	33		23		
TF/SC	N/A	N/A	N/A	N/A		N/A
Design Trickling Filter Loading (kg BOD/m^3/d)					1.6	
Volume of Trickling Filter (m^3) - organic load					51,594	
Depth of Tower (m)					6	
Area of Trickling Filter (m^2)					8,599	
Design AAF Hydraulic Loading m^3/m^2/d-Minimum					45	
Average Hydraulic Loading rate (m^3/m^2.d)					53	
Design MLSS (mg/L) in SC					2000	
MLVSS/MLSS					0.8	
Design F/M (kg BOD/kg MLVSS)					0.28	
Observed Sludge Yield					0.71	
Effective "Solids Retention Time" (days)					6	
Aeration Basin Volume (m^3)-F/M ratio					45,513	
HRT (hr) @ Flow AAF					1.9	
Aeration Basin Depth (m)					4.5	
Surface Area Required (m^2) - F/M ratio					10,114	
Minimum HRT Requirement (hr)					0.7	
Aeration Basin Volume (m^3)- HRT					24,700	
Surface Area Required (m^2) - HRT					5,489	
Return Activated Sludge % (RAS)					75%	
Oxygen Requirement (kg O_2/kg BOD_5)					1.26	
Actual Oxygen Transfer Rate AOTR (t/d)					2.7	
SOTR (t/d O_2)					6.0	
Air requirement (scfm)					1788	
Mixing requirement (m^3 air/m^3/min)					0.015	
Air requirement (scfm)					13722	
Design Effluent BOD Concentration (mg/L)					20	
Final Clarifier			N/A	N/A		N/A
Surface Overflow Rate (m^3/m^2/d) -Max flow	72	72			60	
Surface Area 1 (m^2) -SOR	1,583	3,167			15,200	
Solids Loading Rate (kg/m^2/d)-Max Flow	150	150			150	
Surface Area 2 (m^2) -SLR	109	218			21,280	
Depth (m)	4.5	4.5			4.5	
Volume (m^3)	7,125	14,250			95,760	
HRT (hr) @ Design Flow	1.50	1.50			1.44	
Design Effluent TSS Concentration (mg/L)	20	20			20	

IIWWTP Process Design: Design Case Scenario (Cont'd)

YEAR	Interim 2021				Build-out to Secondary 2036	
Option	Option 1 25% ADWF RTF	Option 2 50% ADWF RTF	Option 3 CEP Only	Option 4 CEP + 50% ADWF RTF no SCL	Option 1 TF/SC	Option 2 BAF
Thickener - Gravity (for PS)						
Raw Primary Sludge (t/d)	58.0	58.0	92.8	92.8	59.5	59.5
Chemical Sludge (t/d)			13.8	13.8		
Total Primary/CEP Sludge (t/d)	58.0	58.0	106.6	106.6	59.5	59.5
Solids Concentration After Thickening (%)	5%	5%	5%	5%	5%	5%
Sludge Volume (m^3/d)	1,160	1,160	2,133	2,133	1,190	1,190
Design Solids Loading MML (kg/m^2/d)	100	100	100	100	100	100
Surface Area (m^2)	580	580	1066	1066	595	595
Thickener - DAF (for WAS)			N/A	N/A		
Waste Activated Sludge (t/d) WAS	14.0	28.1			50.5	66.8
Solids Concentration After DAF (%)	3.5%	3.5%			3.5%	3.5%
Sludge Volume (m^3/d)	401	802			1442	1909
Design Solids Loading (kg/m^2/d)	48.0	48.0			48.0	48.0
Surface Area (m^2)	293	585			1,052	1,392
Digester						
Digester VSS Loading (kg/d/m^3)	2.5	2.5	2.5	2.5	2.5	2.5
Sludge VSS/TSS Ratio	88%	88%	85%	85%	88%	88%
Digester Volume (m^3) by VSS loading	25,359	30,301	36,256	36,256	38,711	44,466
Un-digested dry tonnes (T/d)	72	86	107	107	110	126
Digested dry tonnes (T/d)	40	48	61	61	62	71
Design HRT (d)	20	20	20	20	20	20
Digested Sludge Solids (%)	2.6%	2.5%	2.9%	2.9%	2.3%	2.3%
VS destruction %	50%	50%	50%	50%	50%	50%
Digested sludge VSS (T/d)	32	38	45	45	48	56
Digested sludge VSS/TSS ratio	79%	79%	74%	74%	79%	79%
Digested Sludge Volume (m^3/d) (without dewatering)	1,561	1,962	2,133	2,133	2,632	3,099
Sludge Cake (%) (dewatered)	30%	30%	35%	35%	30%	30%
Sludge Cake (m^3/d) (dewatered)	134	161	175	175	205	236
Actual HRT (d)	16	15	17	17	15	14
Digester Volume (m^3) by Design HRT	31,224	39,247	42,655	42,655	52,642	61,985
Required volume (m^3) (max)	31,224	39,247	42,655	42,655	52,642	61,985
Dewatering						
Centrifuge (L/min)	1,200	1,200	1,200	1,200	1,200	1,200
Days of Operation / week	5	5	5	5	5	5
Hours of operation / day	7	7	7	7	7	7
No. Centrifuges	4.3	5.5	5.9	5.9	7.3	8.6
Sludge Cake (%) (dewatered)	30%	30%	35%	35%	27%	27%
Sludge Cake (m3/d) (dewatered)	134	161	175	175	228	262
Pressate Treatment SBR (optional)						
Pressate volume (m3/d)	1,427	1,802	1,958	1,958	2,404	2,837
SBR Volume (m3/d) = 1.8 x Pressate vol.	2,568	3,243	3,524	3,524	4,327	5,107
Depth of Reactor (m)	4.5	4.5	4.5	4.5	4.5	4.5
Area of Reactor (m2/d)	571	721	783	783	962	1,135
Estimated Effluent @ Max Flow						
BOD (mg/L)	51	48	36	34		
SS (mg/L)	37	35	15	28		
Estimated Effluent @ AAF/ 2*ADWF						
BOD (mg/L)	114	93	94	64	20	20
SS (mg/L)	83	68	39	62	20	20

LGWWTP Process Design: Design Case Scenario

	INTERIM			BUILD-OUT
OPTION	**1**	**2A**	**2B**	**3**
YEAR	**2031**	**2031**	**2031**	**2046**
	CEP Only	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Average Dry Weather Flow (ML/d), ADWF	104	104	104	111
Average Annual Flow (ML/d), AAF	125	125	125	133
Peak Wet Weather Flow (ML/d), PWWF	337	337	337	356
Maximum Month BOD Loading (t/d), MM BOD	25	25	25	28
Maximum Month TSS Loading (t/d), MM TSS	28	28	28	32
Average Annual BOD Loading (t/d), AA BOD	19	19	19	21
Average Annual TSS Loading (t/d), AA TSS	20	20	20	22
Primary Clarifier				
Average Annual Flow (ML/d)	125	125	125	133
Peak Wet Weather Flow (ML/d)	337	337	337	356
Overflow Rate ($m^3/m^2/d$) - AAF	50	50	50	50
Overflow Rate ($m^3/m^2/d$) - PWWF	130	130	130	130
Surface Area (m^2) - AAF	2,496	2,496	2,496	2,664
Surface Area (m^2) - PWWF	2,592	2,592	2,592	2,738
Depth (m)	2.79	2.79	2.79	2.79
Volume (m^3) - AAF	6,964	6,964	6,964	7,433
Volume (m^3) - PWWF	7,233	7,233	7,233	7,640
Raw Influent BOD Loading (t/d)	25	25	25	28
Raw Influent TSS Loading (t/d)	28	28	28	32
Total Influent BOD Loading (t/d)	25	25	25	28
Total Influent TSS Loading (t/d)	28	28	28	32
PC Influent MM BOD Conc. @ AAF (mg/L)	200	200	200	210
PC Influent MM TSS Conc. @ AAF (mg/L)	224	224	224	240
Design PC BOD removal (%)	55%	28%	55%	27%
Design PC TSS removal (%)	80%	61%	80%	60%
PC Effluent BOD Loading (t/d)	11	18	11	20
PC Effluent TSS Loading (t/d)	6	11	6	13
PC Effluent BOD Conc. @ AAF (mg/L)	90	144	90	153
PC Effluent TSS Conc. @ AAF(mg/L)	45	88	45	96
Chemical Usage		**N/A**		**N/A**
Alum Dosage (mg/L)	70		70	
Polymer Dosage (mg/L)	0.5		0.5	
Alum($Al_2(SO_4)_3$) Volume - AAF (m^3/d)	13.3		13.3	
Polymer Volume - AAF (m^3/d)	0.2		0.2	
Alum($Al_2(SO_4)_3$) Volume - PWWF (m^3/d)	35.9		35.9	
Polymer Volume - PWWF(m^3/d)	0.4		0.4	
Biological Treatment	**N/A**			
Plant Capacity % of ADWF	0.0001	50%	50%	200%
Design Flow (MLD)		52	52	222
Treating % of MM BOD / TSS loading		50%	50%	100%
Design BOD Loading (t/d)		9.0	5.6	20.4

LGWWTP Process Design: Design Case Scenario (Cont'd)

	INTERIM			**BUILD-OUT**
OPTION	**1**	**2A**	**2B**	**3**
YEAR	**2031**	**2031**	**2031**	**2046**
	CEP Only	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
BAF	**N/A**			
Sludge Yield		0.94	0.94	0.94
Sludge Age (days)		2	2	2
Design BOD Loading (kg/m^3/d)		4.50	4.50	5.50
Design TSS Loading (kg/m^3/d)		4.60	4.60	4.60
Design Hydraulic Loading m^3/m^2/d-average		144	144	144
Design Hydraulic Loading m^3/m^2/d-peak		144	144	240
Backwash flow Ml/d (10% of flow treated)		6	6	13
Reactor Area (m^2) -TSS		321	165	752
Reactor Area (m^2) - BOD		541	338	1004
Reactor Area (m^2) - average flow		404	404	863
Reactor Area (m^2) - peak flow		404	404	981
Depth of media (m)		3.7	3.7	3.7
Volume Required (m^3) - based on Max Reactor Area		2000	1496	3716
Oxygen Requirement (kg O_2/kg BOD_5)		1.60	1.60	1.6
Actual Oxygen Transfer Rate AOTR (t/d)		12.7	7.3	25.6
SOTR (t/d O2)		28.0	16.1	56.3
Air requirement (sCFM)		8406	4842	16896
Design Effluent BOD Concentration (mg/L)		20	20	20
Design Effluent TSS Concentration (mg/L)		20	20	20
Thickener - Gravity (for PS) insert Y or N	Y	Y	Y	Y
Raw Primary Sludge (t/d) MML	22.4	17.1	22.4	19.2
Chemical Sludge (t/d) MML	2.9		2.9	
Total Primary/CEP Sludge MML (t/d)	25.3	17.1	25.3	19.2
Raw Primary Sludge (t/d) AAL	15.7	11.9	15.7	13.4
Chemical Sludge (t/d) AAL	2.9		2.9	
Total Primary/CEP Sludge AAL (t/d)	18.6	11.9	18.6	13.4
Solids Concentration After Thickening (%)	5%	5%	5%	5%
Sludge Volume MML (m^3/d)	506	342	506	384
Sludge Volume AAL (m3/d)	372	239	372	269
Design Solids Loading MML (kg/m^2/d)	100	100	100	100
Surface Area (m^2)	253	171	253	192
Thickener - DAF (for WAS or Combined Primary)				
Sludge (t/d)		7.5	4.3	17.1
Chemical Sludge (t/d)				
Total sludge MML (t/d)		7.5	4.3	17.1
Sludge (t/d) AAL		11.6	6.9	12.3
Chemical Sludge (t/d) AAL				
Total sludge AAL (t/d)		11.6	6.9	12.3
Solids Concentration After DAF (%)	3.5%	3.5%	3.5%	3.5%
Sludge Volume MML(m^3/d)		214	123	489
Sludge Volume AAL (m3/d)		333	198	350
Design Solids Loading (kg/m^2/d) 48 for WAS 96 for Co-DAF	48.0	48.0	48.0	48.0
Surface Area (m^2)		156	90	357
Digester				
Digester VSS Loading (kg/d/m^3)	2.2	2.2	2.2	2.2
Sludge VSS/TSS Ratio	90%	85%	87%	80%
Digester Volume (m^3) by VSS loading	10,355	9,490	11,714	13,210
Un-digested dry tonne MML (T/d)	25	25	30	36
Un-digested dry tonne AAL (T/d)	19	24	25	26
Digested dry tonne MML (T/d)	11	12	14	22
Digested dry tonne AAL (T/d)	8	12	12	15
Design HRT (d)	15	15	15	15
Digested Sludge Solids (%)	2.1%	2.2%	2.2%	2.5%
VS destruction %	65%	60%	60%	50%
Digested Sludge Volume MML (m^3/d)	506	555	629	873
Digested Sludge Volume AAL (m^3/d)	372	572	569	619
HRT based on VSS Loading (d)	20	17	19	15
Digester Volume (m^3) by Design HRT	7,594	8,331	9,441	13,100
Required Volume (Max) m^3	10,355	9,490	11,714	13,210

LGWWTP Process Design: Design Case Scenario (Cont'd)

	INTERIM			BUILD-OUT
OPTION	**1**	**2A**	**2B**	**3**
YEAR	**2031**	**2031**	**2031**	**2046**
	CEP Only	50% BAF (No CEP)	CEP+50% BAF	2 x ADWF BAF
Dewatering				
Centrifuge (L/min)	900	900	900	900
Days of Operation / week	5	5	5	5
Hours of operation / day	14	21	21	14
No. Centrifuges	0.9	0.7	0.8	1.6
Sludge Cake (%) (dry tonnes)	35%	27%	27%	27%
Sludge Cake (m3/d) (MML, dewatered)	30	45	52	81
Sludge Cake (m3/d) (AAL, dewatered)	22	43	45	57
Pressate Treatment SBR				
Pressate volume (m3/d)	476	511	577	793
SBR Volume (m3/d) = 1.8 x Pressate vol.	857	919	1,038	1,427
Depth of Reactor (m)	4.5	4.5	4.5	4.5
Area of Reactor (m2)	190	204	231	317
Effluent Standard				
BOD mg/l	130	130	130	45
TSS mg/l	130	130	130	45
Estimated Effluent @ Max Flow				
BOD (mg/L)	33	30	20	12
SS (mg/L)	17	19	11	12
Estimated Effluent @ AAF/ 2*ADWF				
BOD (mg/L)	90	80	53	20
SS (mg/L)	45	52	31	20

APPENDIX B: CAPITAL COST ESTIMATES (DESIGN CASE)

IIWWTP Capital Cost Estimates: Interim Upgrade (2021 Design Flow) Option 1, Primary + 25% ADWF RTF

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$2,000,000
Fill (5 m)			350,000	m^3	$10	$3,500,000
Preloading (1.5 m & 4 m)			165,000	m^3	$15	$2,475,000
Ground Densification - new			80,000	m^3	$8	$640,000
Ground Densification - exist			-	m^3	$8	$0
Soil Anchors - existing			-	each	$4,000	$0
Dewatering				l.s.		$250,000
Total for Site Improvement						$8,865,000
Roughing Trickling Filter	1	9,125	9,125	m^3	$920	$8,395,000
Odour Control (allowance)						$500,000
Solids Contact	0	14,190	-	m^3	$405	$0
Secondary Clarifiers	2	1,320	2,640	m^2	$2,140	$5,649,600
Gravity Thickeners	0	308	-	m^2	$4,500	$0
Sludge Blending tank						$250,000
DAF Thickeners	1	363	363	m^2	$21,200	$7,695,600
Digesters	1	8,525	8,525	m^3	$940	$8,013,500
Mechanical Dewatering				l.s.		$7,000,000
Site Works:						
Diversion Channel to RTF						$200,000
Pumping to Bioreactor			300	HP	$3,000	$900,000
Roads/Grading				l.s.		$200,000
750 mm RAS			300	m	$500	$150,000
600 mm WAS			325	m	$450	$146,250
2400 mm Effluent Tunnel			650	m	$1,925	$1,251,250
Admin/Maint. Building	0	5,000	-	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$1,855,000
Expansion of Cogeneration				l.s.		$1,500,000
Existing Facility Upgrades:						
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$63,571,200**
Division 1 Cost		2.5%				$1,368,000
Engineering		16%				$10,171,000
Project Management/ Quality Control		4%				$2,543,000
Contingency		30%				$19,071,000
Total (Capital Costs)						**$96,724,200**

IIWWTP Capital Cost Estimates: Interim Upgrade (2021 Design Flow) Option 2, Primary + 50% ADWF RTF

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			665,000	m^3	$10	$6,650,000
Preloading (1.5 m & 4 m)			315,000	m^3	$15	$4,725,000
Ground Densification - new			150,000	m^3	$8	$1,200,000
Ground Densification - exist			-	m^3	$8	$0
Soil Anchors - existing			-	each	$4,000	$0
Dewatering				l.s.		$1,200,000
Total for Site Improvement						$21,775,000
Roughing Trickling Filter	2	9,125	18,250	m^3	$920	$16,790,000
Odour Control (allowance)						$1,000,000
Solids Contact	0	14,190	-	m^3	$405	$0
Secondary Clarifiers	4	1,320	5,280	m^2	$2,140	$11,299,200
Gravity Thickeners	0	308	-	m^2	$4,500	$0
Sludge Blenging tank						$500,000
DAF Thickeners	2	363	726	m^2	$21,200	$15,391,200
Digesters	2	8,525	17,050	m^3	$940	$16,027,000
Mechanical Dewatering				l.s.		$10,000,000
Site Works:						
Diversion Channel to RTF						$300,000
Pumping to Bioreactor			360	HP	$3,000	$1,080,000
Roads/Grading				l.s.		$400,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			910	m	$450	$409,500
2400 mm Effluent Tunnel			760	m	$1,925	$1,463,000
Admin/Maint Building	0	5,000	-	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$3,711,000
Expansion of Cogeneration				l.s.		$7,000,000
Existing Facility Upgrades:						
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$118,495,900**
Division 1 Cost		2.5%				$2,418,000
Engineering		16%				$18,959,000
Project Management/ Quality Control		4%				$4,740,000
Contingency		30%				$35,549,000
Total (Capital Costs)						**$180,161,900**

IIWWTP Capital Cost Estimates: Interim Upgrade (2021 Design Flow) Option 3, CEP Only

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$0
Fill (5 m)			115,000	m^3	$10	$1,150,000
Preloading (1.5 m & 4 m)			90,000	m^3	$15	$1,350,000
Ground Densification - new			35,000	m^3	$8	$280,000
Ground Densification - exist			-	m^3	$8	$0
Soil Anchors - existing			-	each	$4,000	$0
Dewatering				l.s.		$250,000
Total for Site Improvement						$3,030,000
Roughing Trickling Filter	2	9,125	-	m^3	$920	$0
Odour Control (allowance)						$0
Solids Contact	0	14,190	-	m^3	$405	$0
Secondary Clarifiers	0	1,320	-	m^2	$2,140	$0
Gravity Thickeners	2	308	616	m^2	$4,500	$2,772,000
Sludge Blenging tank						$0
DAF Thickeners	0	363	-	m^2	$21,200	$0
Digesters	3	8,520	25,560	m^3	$940	$24,026,400
Mechanical Dewatering				l.s.		$10,000,000
Chemical Feed System						$1,500,000
Site Works:						
Diversion Channel to RTF						$0
Pumping to Bioreactor			-	HP	$3,000	$0
Roads/Grading				l.s.		$150,000
750 mm RAS			-	m	$500	$0
600 mm WAS			-	m	$450	$0
2400 mm Effluent Tunnel			-	m	$1,925	$0
Admin/Maint. Building	0	5,000	-	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$1,593,000
Expansion of Cogeneration				l.s.		$9,900,000
Existing Facility Upgrades:						
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$63,971,400**
Division 1 Cost		2.5%				$1,524,000
Engineering		16%				$10,235,000
Project Management/ Quality Control		4%				$2,559,000
Contingency		30%				$19,191,000
Total (Capital Costs)						**$97,480,400**

IIWWTP Capital Cost Estimates: Interim Upgrade (2021 Design Flow) Option 4, CEP + 50% ADWF RTF no Secondary Clarifier

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$2,500,000
Fill (5 m)			420,000	m^3	$10	$4,200,000
Preloading (1.5 m & 4 m)			315,000	m^3	$15	$4,725,000
Ground Densification - new			87,000	m^3	$8	$696,000
Ground Densification - exist			-	m^3	$8	$0
Soil anchors - existing			-	each	$4,000	$0
Dewatering				l.s.		$925,000
Total for Site Improvement						$13,046,000
Trickling Filters	2	9,125	18,250	m^3	$920	$16,790,000
Solids Contact	0	14,190	-	m^3	$415	$0
Odour Control (allowance)						$1,000,000
Secondary Clarifiers	0	1,520	-	m^2	$2,140	$0
Gravity Thickeners	2	215	430	m^2	$4,500	$1,935,000
DAF Thickeners	0	363	-	m^2	$21,200	$0
Digesters	3	8,520	25,560	m3	$940	$24,026,400
Mechanical Dewatering				l.s.		$10,000,000
Chemical Feed System						$1,500,000
Site Works:						
Diversion Channel to RTF						$300,000
Pumping to Bioreactor			360	HP	$3,000	$1,080,000
Roads/Grading				l.s.		$250,000
750 mm RAS			700	m	$500	$350,000
600 mm WAS			-	m	$450	$0
2400 mm Effluent			600	m	$1,925	$1,155,000
Admin/Maint Building		-	-	m^2	$1,600	$0
Control System (allowance)		4%		l.s.		$2,150,000
Expansion of Cogeneration				l.s.		$9,900,000
Existing Facility Upgrades:						
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$94,482,400**
Division 1 Cost		2.5%				$2,036,000
Engineering		16%				$15,117,000
Project Management/ Quality Control		4%				$3,779,000
Contingency		30%				$28,345,000
Total (Capital Costs)						**$143,759,400**

IIWWTP Capital Cost Estimates: Build-out to Secondary Upgrade (2036 Design Flow) Option 1, Primary + 100% of 2 x ADWF TF/SC

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$8,000,000
Fill (5 m)			665,000	m^3	$10	$6,650,000
Preloading (1.5 m & 4 m)			315,000	m^3	$15	$4,725,000
Ground Densification - new			150,000	m^3	$8	$1,200,000
Ground Densification - exist plant			210,000	m^3	$8	$1,680,000
Soil Anchors - existing plant			2,220	each	$4,000	$8,880,000
Dewatering allowance				l.s.		$3,700,000
Total for Site Improvement						$34,835,000
Trickling Filters	6	9,125	54,750	m^3	$920	$50,370,000
Odour Control (allowance)						$3,000,000
Solids Contact	4	6,350	25,400	m^3	$415	$10,541,000
Secondary Clarifiers	16	1,320	21,120	m^2	$2,140	$45,196,800
Sludge Blending Tank				l.s.		$1,000,000
DAF Thickeners	3	363	1,089	m^2	$21,200	$23,086,800
Digesters	4	8,525	34,100	m^3	$940	$32,054,000
Mechanical Dewatering				l.s.		$25,800,000
Site Works:						
Pumping to Bioreactor			2,080	HP	$3,000	$6,240,000
Roads/Grading				l.s.		$1,000,000
Diversion Channel to TF				l.s.		$300,000
750 mm RAS			2,800	m	$500	$1,400,000
600 mm WAS			3,640	m	$450	$1,638,000
2400 mm Effluent			750	m	$1,925	$1,444,000
Tunnel				l.s.		$2,000,000
Admin/Maint Building	1	2,500	2,500	m^2	$1,600	$4,000,000
Control System (allowance)		4%		l.s.		$6,610,000
Expansion of Cogeneration				l.s.		$11,700,000
Existing Facility Upgrades:						
New Headwork Upgrade				l.s.		$55,000,000
Seismic Upgrades Exist. Structures				l.s.		$1,000,000
Thermophilic Upgrades of Exist Digester				l.s.		$2,150,000
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$331,365,600**
Division 1 Cost		2.5%				$7,413,000
Engineering		16%				$53,018,000
Project Management/ Quality Control		4%				$13,255,000
Contingency		30%				$99,410,000
Total (Capital Costs)						**$504,461,600**

IIWWTP Capital Cost Estimates: Build-out to Secondary Upgrade (2036 Design Flow) Option 2, Primary + 100% of 2 x ADWF BAF

CAPITAL COSTS	Cell Units	Quantity per Cell	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$2,500,000
Fill (5 m)			280,000	m^3	$10	$2,800,000
Preloading (1.5 m & 4 m)			182,000	m^3	$15	$2,730,000
Ground Densification - new			68,000	m^3	$8	$544,000
Ground Densification - exist plant			210,000	m^3	$8	$1,680,000
Soil Anchors - existing plant			2,220	each	$4,000	$8,880,000
Dewatering Allowance				l.s.		$1,900,000
Total for Site Improvement						$21,034,000
Trickling Filters	0	9,125	-	m^3	$920	$0
BAF						$92,600,000
Odour Control (allowance)						$500,000
Solids Contact	0	6,350	-	m^3	$415	$0
Secondary Clarifiers	0	1,320	-	m^2	$2,140	$0
Sludge Blending Tank				l.s.		$1,000,000
DAF Thickeners	3.3	363	1,198	m^2	$21,200	$25,395,480
Digesters	4.4	8,525	37,510	m^3	$940	$35,259,400
Mechanical Dewatering				l.s.		$25,800,000
Site Works:						
Pumping to Bioreactor			1,000	HP	$3,000	$3,000,000
Roads/Grading				l.s.		$600,000
Diversion Channel to TF						$300,000
750 mm RAS			-	m	$500	$0
600 mm WAS			3,640	m	$450	$1,638,000
2400 mm Effluent			750	m	$1,925	$1,444,000
Tunnel						$1,000,000
Admin/Maint Building	1	2,500	2,500	m^2	$1,600	$4,000,000
Control System (allowance)		4%		l.s.		$6,190,000
Expansion of Cogeneration				l.s.		$11,700,000
Existing Facility Upgrades:						
New Headwork Upgrade				l.s.		$55,000,000
Seismic Upgrades Exist. Structures				l.s.		$1,000,000
Thermophilic Upgrades of Exist Digester				l.s.		$2,150,000
Sludge Pre-treatment				l.s.		$6,000,000
Submerged Effluent Launders				l.s.		$5,000,000
Sub-Total						**$300,610,880**
Division 1 Cost		2.5%				$6,989,000
Engineering		16%				$48,098,000
Project Management/ Quality Control		4%				$12,024,000
Contingency		30%				$90,183,000
Total (Capital Costs)						**$457,904,880**

LGWWTP Capital Cost Estimates: Interim Upgrade (2031 Design Flow) Option 1, CEP Only

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)			1	l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground Densification - New	5364	14	75,096	m^3	$8	$600,768
Ground Densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil Anchors - existing			370	each	$4,000	$1,480,000
Dewatering			2,960	m^2	$100	$296,000
Total for Site Improvement						$4,056,768
Treatment Components:						
Chemical Dosing			1	LS	$500,000	$500,000
Primary Clarifiers	0	186	0	m^2	$4,056	$0
Roughing Trickling Filters	0	0	0	m^3	$900	$0
BAF			0	l.s.	$14,524,084	$0
Gravity Thickeners	1	147	147	m^2	$4,500	$663,351
DAF Thickeners	0	177	0	m^2	$20,905	$0
Digesters	2	3498	6,996	m^3	$1,270	$8,885,222
Mechanical Dewatering (Centrifuge)	0			l.s.	$1,254,277	$0
SBR	0	160	0	m^3	$5,309	$0
UV	-			ML/d PWWF		-
Odour Control	Allowance					$500,000
Site Works:						
Pumping to Bioreactor			0	kW	-	$0
Roads/Grading			1	l.s.	$100,000	$100,000
Piping (1050mm dia.)		1050	250.4	m	$1,050	$262,920
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			10,548,574	%	4.00%	$421,943
Electrical Substation		885	1	l.s.	$65,000	$65,000
Existing Facility Upgrade - Seismic Upgrade of PST roofs			1	l.s.	$200,000	$200,000
Sub-Total						**$16,955,205**
Division 1 Cost		2.5%				$322,461
Engineering		16%				$2,712,833
Project Management/ Quality Control		4%				$678,208
Contingency		30%				$5,086,561
Total (Capital Costs)						**$25,756,000**

LGWWTP Capital Cost Estimates: Interim Upgrade (2031 Design Flow) Option 2A, 50% ADWF BAF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground Densification - New	5364	14	75,096	m^3	$8	$600,768
Ground Densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil Anchors - existing			370	each	$4,000	$1,480,000
Dewatering			2,960	m^2	$100	$296,000
Total for Site Improvement						$4,056,768
Treatment Components:						
Chemical Dosing	Allowance					$500,000
Primary Clarifiers	0	186	0	m^2	$4,056	$0
Roughing Trickling Filters	0	0	0	m^3	$900	$0
BAF			1	l.s.	$14,524,084	$14,524,084
Gravity Thickeners	1	147	147	m^2	$4,500	$663,351
DAF Thickeners	1	177	177	m^2	$20,905	$3,692,278
Digesters	1	3498	3,498	m^3	$1,270	$4,442,611
Mechanical Dewatering (Centrifuge)	0			l.s.	$1,254,277	$0
SBR	0	160	0	m^3	$5,309	$0
UV	-			ML/d PWWF		-
Odour Control	Allowance					$500,000
Site Works:						
Pumping to Bioreactor			89	kW	$9,251	$819,281
Roads/Grading			1	l.s.	$100,000	$100,000
Piping (1050 mm dia.)		1050	662	m	$1,050	$695,100
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			24,322,324	%	4.00%	$972,893
Electrical Substation		1,318	1	l.s.	$85,000	$85,000
Existing Facility Upgrade - Seismic Upgrade of PST roofs			1	l.s.	$200,000	$200,000
Sub-Total						**$32,551,366**
Division 1 Cost		2.5%				$712,365
Engineering		16%				$5,208,219
Project Management/ Quality Control		4%				$1,302,055
Contingency		30%				$9,765,410
Total (Capital Costs)						**$49,540,000**

LGWWTP Capital Cost Estimates: Interim Upgrade (2031 Design Flow) Option 2B, CEP + 50% ADWF BAF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground Densification - New	5364	14	75,096	m^3	$8	$600,768
Ground Densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil Anchors - existing			370	each	$4,000	$1,480,000
Dewatering			2,960	m^2	$100	$296,000
Total for Site Improvement						$4,056,768
Treatment Components:						
Chemical Dosing	Allowance					$500,000
Primary Clarifiers	0	186	0	m^2	$4,056	$0
Roughing Trickling Filters	0	0	0	m^3	$900	$0
BAF			1	l.s.	$14,524,084	$14,524,084
Gravity Thickeners	1	147	147	m^2	$4,500	$663,351
DAF Thickeners	1	177	177	m^2	$20,905	$3,692,278
Digesters	2	3498	6,996	m^3	$1,270	$8,885,222
Mechanical Dewatering (Centrifuge)	0			l.s.	$1,254,277	$0
SBR	0	160	0	m^3	$5,309	$0
UV	-			ML/d PWWF		-
Odour Control	Allowance					$500,000
Site Works:						
Pumping to Bioreactor			89	kW	$9,251	$819,281
Roads/Grading			1	l.s.	$100,000	$100,000
Piping (1050 mm dia.)		1050	662	m	$1,050	$695,100
Admin/Maint Building			1	l.s	$1,300,000	$1,300,000
Control System			28,764,935	%	4.00%	$1,150,597
Electrical Substation		1,193	1	l.s.	$75,000	$75,000
Existing Facility Upgrade - Seismic Upgrade of PST roofs			1	l.s.	$200,000	$200,000
Sub-Total						**$37,161,682**
Division 1 Cost		2.5%				$827,623
Engineering		16%				$5,945,869
Project Management/ Quality Control		4%				$1,486,467
Contingency		30%				$11,148,505
Total (Capital Costs)						**$56,571,000**

LGWWTP Capital Cost Estimates: Build-out to Secondary Upgrade (2046 Design Flow) Option 3, 2×ADWF BAF

CAPITAL COSTS	# of units	Quantity per unit	Total Quantity	Units	Price/Unit	Amount
Site Improvements:						
Remove Sludge Stockpile (allow.)				l.s.		$0
Fill (5 m)			0	m^3	$10	$0
Preloading (1.5 m & 4 m)			0	m^3	$15	$0
Ground Densification - New	7822.56	14	109,516	m^3	$8	$876,127
Ground Densification - Burrard Inlet Berm	15000	14	210,000	m^3	$8	$1,680,000
Soil Anchors - existing			370	each	$4,000	$1,480,000
Dewatering			4,300	m^2	$100	$430,000
Total for Site Improvement						$4,466,127
Treatment Components:						
Chemical Dosing			0		$500,000	$0
Primary Clarifiers	0	186	0	m^2	$4,056	$0
Roughing Trickling Filters	0	0	0	m^3	$900	$0
BAF			1	l.s.	$23,649,647	$23,649,647
Gravity Thickeners	1	147	147	m^2	$4,500	$663,351
DAF Thickeners	2	177	357	m^2	$20,905	$7,458,941
Digesters	2	3498	6,996	m^3	$1,270	$8,885,222
Mechanical Dewatering (Centrifuge)	1			l.s.	$1,254,277	$1,254,277
SBR				m^3		$0
UV			222	ML/d PWWF	$10,000	$2,220,000
Odour Control	Allowance					$500,000
Site Works:						
Pumping to Bioreactor			378	kW	$6,529	$2,468,600
Roads/grading			1	l.s.	$100,000	$100,000
Piping (1600 mm dia.)		1600	687	m	$1,600	$1,099,200
Admin/Maint Building			1	l.s	$2,000,000	$2,000,000
Control System			44,631,438	%	4.00%	$1,785,258
Electrical Substation		2,203	1	l.s.	$115,000	$115,000
Existing Facility Upgrade - Seismic Upgrade of PST roofs			1	l.s.	$200,000	$200,000
Existing Facility Upgrade - New Headwork Upgrade			1	l.s.	$14,000,000	$14,000,000
Sub-Total						**$70,865,623**
Division 1 Cost		2.5%				$1,659,987
Engineering		16%				$11,338,500
Project Management/ Quality Control		4%				$2,834,625
Contingency		30%				$21,259,687
Total (Capital Costs)						**$107,959,000**

APPENDIX C: OPERATIONAL CERTIFICATES

PROVINCE OF
BRITISH COLUMBIA

BRITISH COLUMBIA

Environmental Protection
10470 - 152 Street
Surrey, British Columbia
V3R 0Y3
Telephone: (604) 582-5200
Fax: (604) 584-9751

MINISTRY OF WATER,
LAND AND AIR PROTECTION

OPERATIONAL CERTIFICATE
ME-00023

*Under the Provisions of the Waste Management Act
and in accordance with the Greater Vancouver
Regional District Liquid Waste Management Plan*

Greater Vancouver Sewerage and Drainage District

4330 Kingsway

Burnaby, British Columbia V5H 4G8

shall operate a municipal waste water treatment plant located on Iona Island, Richmond, British Columbia, subject to the conditions listed below. Contravention of any of these conditions is a violation of the *Waste Management Act* and may result in prosecution.

1. AUTHORIZED DISCHARGE

1.1 This section applies to the discharge of effluent from **IONA ISLAND WASTE WATER TREATMENT PLANT SERVING THE CITY OF VANCOUVER, UNIVERSITY ENDOWMENT LANDS, PORTIONS OF BURNABY AND RICHMOND (TWIGG AND MITCHELL ISLANDS) AND TRUCKED WASTES MAINLY WITHIN THE GREATER VANCOUVER REGIONAL DISTRICT AND OCCASIONALLY FROM OTHER AREAS TO GEORGIA STRAIT.** The site reference number for this discharge is E100992.

1.1.1 The maximum daily authorized rate of discharge is 1,530,000 cubic metres/day.

R.H. Robb
Assistant Regional Waste Manager

1.1.2 The maximum daily (flow proportioned 24-hour composite) concentration level of the discharge shall be:

5-day biochemical oxygen demand (BOD_5)	130. mg/L, maximum
Total suspended solids (nonfilterable residue) (TSS)	100. mg/L, maximum

1.1.3 The maximum daily discharge loadings* for BOD_5 and TSS for the final effluent and to be used for the calculation of annual operational certificate fees shall be:

*daily discharge loading is the total amount of contaminants discharged per day (contaminant concentration x rate of discharge)

Year	BOD_5 (tonnes/day)	TSS (tonnes/day)
2004	78.1	72.0
2005	80.0	74.0
2006	82.0	76.0
2007 and thereafter	84.0	78.0

1.1.4 The designated treatment works, approximately located as shown on attached Site Plans A and B, are:

- influent screens;
- preaeration and grit removal facilities;
- primary sedimentation tanks; and
- a submerged outfall having twin 505 metre long parallel diffusers, located approximately 7.2 kilometres offshore and discharging to depths ranging from approximately 72 to 106 metres below mean low water.

1.1.5 The location of the facilities from which the discharge originates is Lot 236, Group 1, NWD.

R.H. Robb
Assistant Regional Waste Manager

1.1.6 The location of the point of discharge is the marine waters of Georgia Strait approximately 2000 metres west of Lot 68291, Group 1, NWD.

2. GENERAL REQUIREMENTS

2.1 Maintenance of Works

The operational certificate holder shall inspect the authorized works regularly and maintain them in good working order. Notify the Regional Waste Manager of any malfunction of these works.

2.2 Bypasses

The discharge of effluent which has bypassed the designated treatment works is prohibited unless the approval of the Regional Waste Manager is obtained and confirmed in writing.

2.3 Emergency Procedures

In the event of an emergency which prevents compliance with a requirement of this operational certificate, that requirement will be suspended for such time as the emergency continues or until otherwise directed by the Regional Waste Manager provided that:

a. Due diligence was exercised in relation to the process, operation or event which caused the emergency and that the emergency occurred notwithstanding this exercise of due diligence;

b. The manager is immediately notified of the emergency; and

c. It can be demonstrated that everything possible is being done to restore compliance in the shortest possible time.

Notwithstanding (a), (b), and (c) above, the manager may require the operation to be suspended or production levels to be reduced to protect the environment while the situation is corrected.

2.4 Process Modifications

The Regional Waste Manager shall be notified prior to implementing changes to any process that may adversely affect the quality and/or quantity of the discharge.

2.5 Trucked Wastes

The operational certificate holder shall not accept Special Wastes as defined in the Special Waste Regulation under the Waste Management Act for disposal at the treatment plant. Tests shall be conducted as deemed necessary to ensure that unacceptable wastes are identified.

2.6 Facility Classification and Operator Certification

The operational certificate holder shall have the works authorized by this permit classified by the Environmental Operators Certification Program Society (Society). The works shall be operated and maintained by persons certified within and according to the program provided by the Society. Certifications must be completed to the satisfaction of the Regional Waste Manager. In addition, the manager shall be notified of the classification level of the facility and certification level of the operator with the highest certification level of the facility and changes of the operator and/or operator certification level of the operator with the highest certification level of the facility.

Alternatively, the works authorized by this operational certificate shall be operated and maintained by persons who the operational certificate holder can demonstrate, to the satisfaction of the Director, are qualified in the safe and proper operation of the facilities for the protection of the environment.

2.7 Sludge Wasting and Disposal or Utilization

Efforts should be taken to beneficially utilize the sludge wasted from the treatment plant. Utilization or disposal of the sludge shall be done in a manner satisfactory to the Regional Waste Manager, or as authorized by regulation under the *Waste Management Act*.

2.8 Posting of Outfall

A sign shall be erected along the alignment of the outfall above the high water mark. The sign shall identify the nature of the works. The wording and size of the sign shall be acceptable to the Regional Waste Manager.

R.H. Robb
Assistant Regional Waste Manager

3. MONITORING AND REPORTING REQUIREMENTS

3.1 Discharge Monitoring

3.1.1 Sampling and Analyses

Suitable sampling facilities shall be installed and maintained and composite or grab samples of the effluent authorized by section 1.1 shall be obtained for analyses as indicated below. A composite sample is to consist of a sample composited in proportion to flow over a 24 hour period (or approved flow proportional continuous sampler may be used). Proper care should be taken in sampling, storing and transporting the samples to adequately control temperature and avoid contamination, breakage, etc.

Parameter	Frequency	Sampling Type	Required Detection Limit
TSS, mg/L	5 times/week	composite	5 mg/L
BOD_5*, mg/L	5 times/week	composite	10 mg/L
Ammonia, nitrogen, mg/L	2 times/month	grab	1.0 mg/L
Fish bioassay (rainbow trout), 96 hour LC50, %	monthly	grab	95% confidence limits

*COD may be used in place of BOD_5 if BOD_5 is examined with COD every fifth sample.

3.1.2 Additional Sampling Parameters

By June 30, 2004, the operational certificate holder will submit to the Regional Waste Manager for approval, a list of additional substances that will be monitored in the effluent and has been reviewed by the Environmental Monitoring Committee. The list shall include the substance name, sampling frequency, sample type and required detection limit. The list of substances shall be reviewed every five years in consultation with the Environmental Monitoring Committee and an updated list shall be submitted to the Regional Waste Manager for approval.

R.H. Robb
Assistant Regional Waste Manager

3.1.3 Toxicity Failures

If the monthly bioassay test fails, the operational certificate holder will conduct a Toxicity Identification Evaluation (TIE) study for the purpose of determining the probable cause of the failure. The results of the failed monthly bioassay test and TIE study will be submitted to the Regional Waste Manager by the end of the month following the month that the bioassay test failure occurred.

3.1.4 Flow Measurement

Provide and maintain a suitable flow measuring device and record once per day the effluent volume discharged over a 24-hour period.

3.2 Monitoring Procedures

3.2.1 Sampling Procedures

Sampling is to be carried out in accordance with procedures described in the latest version of "British Columbia Field Sampling Manual for Continuous Monitoring plus the Collection of Air, Air-Emission, Water, Wastewater, Soil, Sediment, and Biological Samples, 2003 Edition (Permittee)," or by suitable alternative procedures as authorized by the Regional Waste Manager.

A copy of the above manual may be purchased from Queen's Printer Publications Centre, P. O. Box 9452, Stn. Prov. Govt. Victoria, British Columbia, V8W 9V7 (1-800-663-6105 or (250) 387-4609). A copy of the manual is also available for inspection at all Environmental Protection offices.

R.H. Robb
Assistant Regional Waste Manager

3.2.2 Chemical Analyses

Analyses are to be carried out in accordance with procedures described in the latest version of "British Columbia Laboratory Methods Manual for the Analysis of Water, Wastewater, Sediment and Biological Materials and Discrete Ambient Air Sample (March 2003 Permittee Edition)", or by suitable alternative procedures as authorized by the Regional Waste Manager.

A copy of the above manual may be purchased from Queen's Printer Publications Centre, P. O. Box 9452, Stn. Prov. Govt. Victoria, British Columbia, V8W 9V7 (1-800-663-6105 or (250) 387-4609). A copy of the manual is also available for inspection at all Environmental Protection offices.

3.2.3 Quality Assurance

All data analyses required to be submitted by the operational certificate holder shall be conducted by a laboratory acceptable to the Regional Waste Manager. At the request of the manager, the operational certificate holder shall provide the laboratory quality assurance data, associated field blanks, and duplicate analysis results along with the submission of data required under Section 3. of the operational certificate.

3.3 <u>Trucked Wastes Recording</u>

3.3.1 Records

The operational certificate holder shall maintain up to date records in hard copy or electronic format of domestic and commercial sludges which are delivered for disposal at the treatment plant. The records shall be available for inspection and shall include:

Date received or rejected;
Source(s) of waste;
Type of waste (general description);
Estimated quantity of waste, m^3;
Name of carrier.

R.H. Robb
Assistant Regional Waste Manager

3.3.2 Summary of Records

The operational certificate holder shall compile a monthly summary of the records specified in Section 3.3.1. The summary shall include:

Total domestic discharge volume, m^3;
Total number of domestic generator loads;
Total non-domestic discharge volume, m^3;
Total number of non-domestic generator loads;
Total discharge volume, m^3;
Total number of carrier loads.

3.3.3 Rejected Waste Loads

The operational certificate holder shall submit to the Regional Waste Manager, by facsimile **by the end of the next business day,** information on waste loads that have been rejected. The information shall include generator name, contact name and phone number, type of waste, volume and reason for rejecting waste load.

3.4 Outfall Inspection

The operational certificate holder shall have the outfall inspected once every ten years by independent qualified personnel to ensure that it is in good condition. An inspection report shall be submitted to the Regional Waste Manager within 30 days after the inspection date. The first report shall be submitted by **January 31, 2011**.

3.5 Reporting

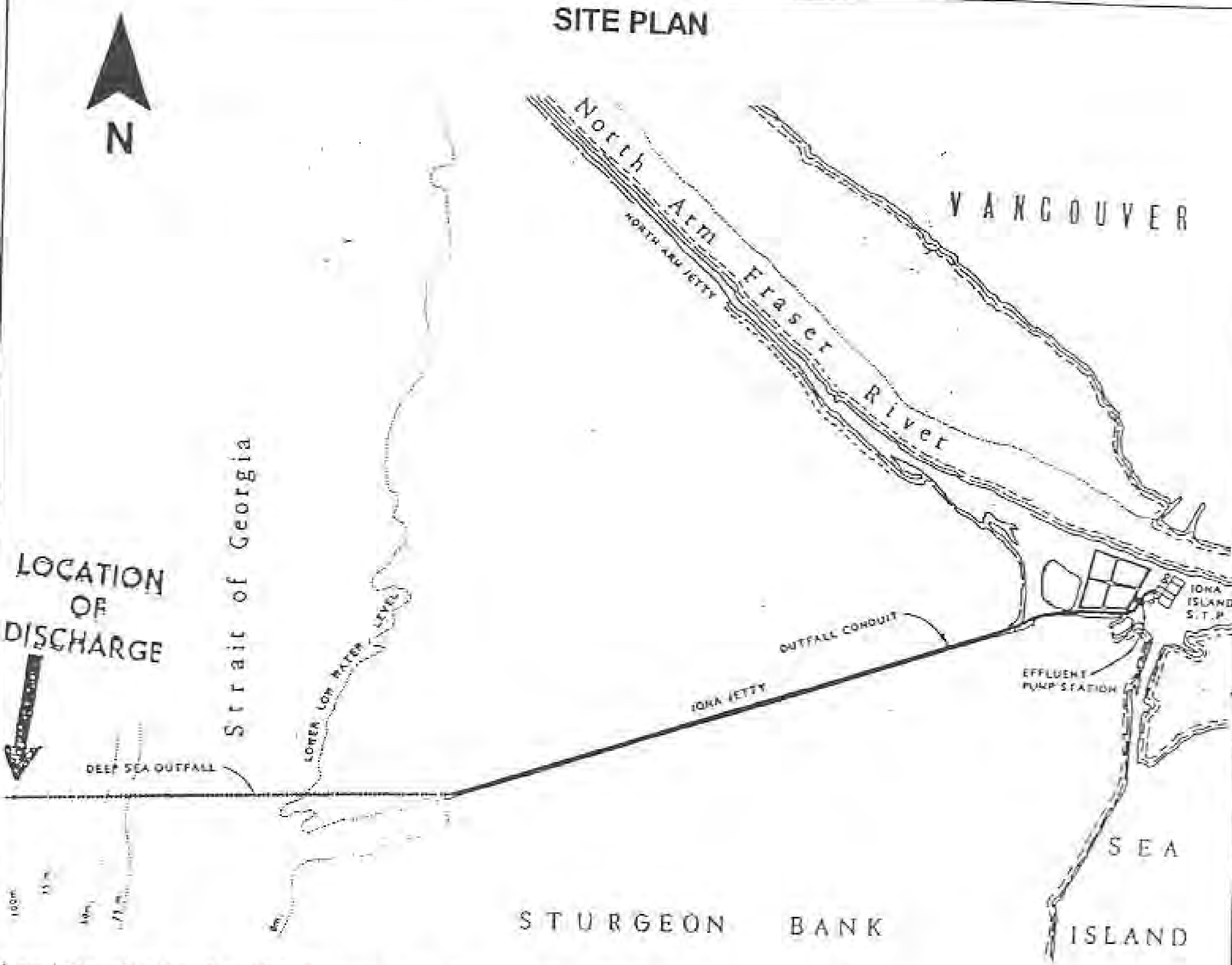
Maintain data of analyses flow measurements and summary of trucked wastes, suitably tabulated, for inspection and post the data monthly on the operational certificate holder's Internet web site. Notify the Regional Waste Manager monthly of any data that is in noncompliance with requirements of this operational certificate.

The following information shall also be included in the liquid waste management plan biennial report:

- trends of the analytical data over the past two years and comparisons with data from the past years; and
- a summary of analytical data submitted in the monthly reports.

R.H. Robb
Assistant Regional Waste Manager

SITE PLAN



Legal Description: Lot 236, Group 1, New Westminster District.

Location Map

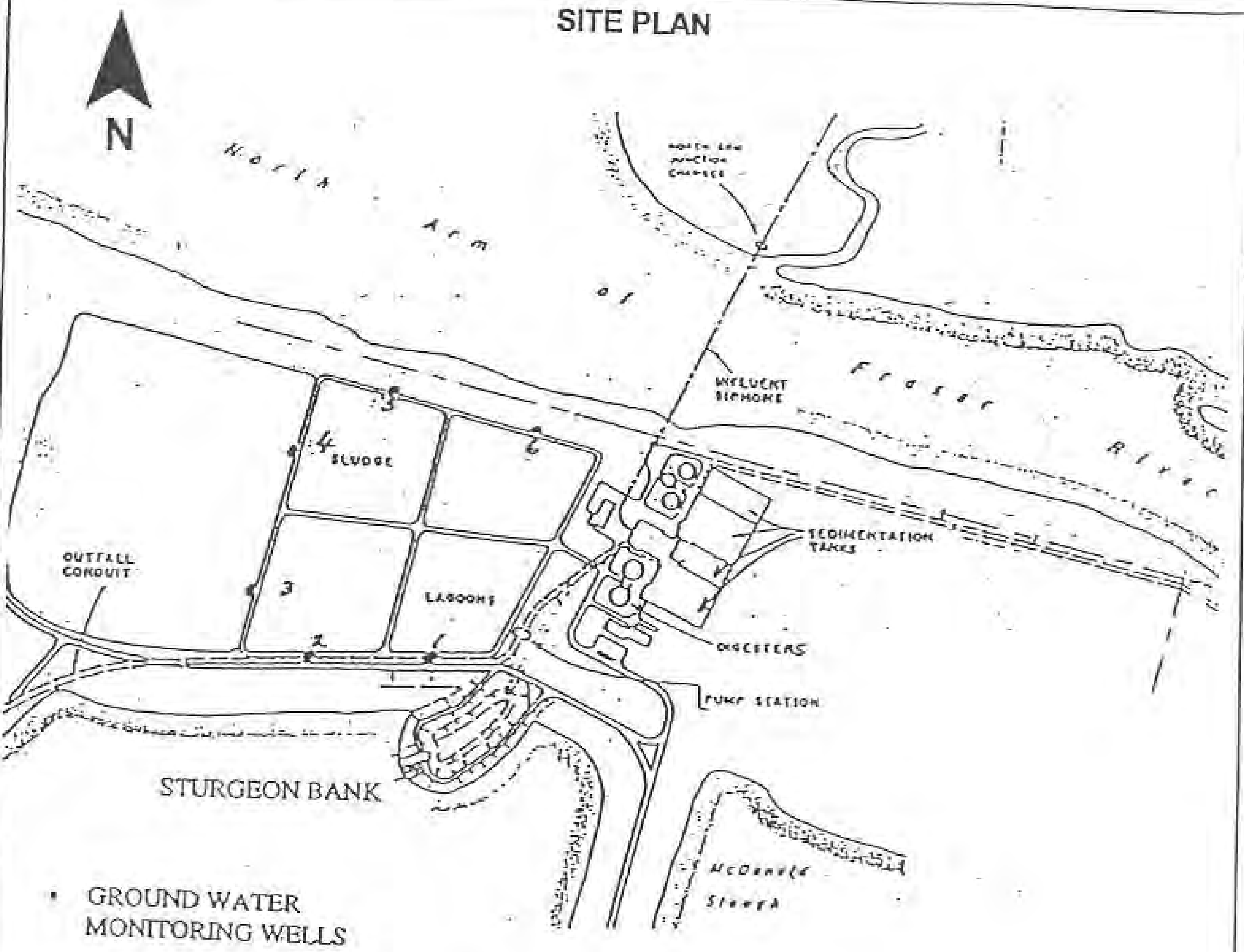
Scale: 1:40,000

Site Plan: A | OC ME-00023

Date: [illegible]

R.H. Robb
Assistant Regional Waste Manager
Lower Mainland Region

SITE PLAN



Legal Description: Lot 236, Group 1, New Westminster District.

Scale: 1:70,700	
Site Plan: B	OC ME-00023
Date:	

R.H. Robb
Assistant Regional Waste Manager
Lower Mainland Region

PROVINCE OF
BRITISH COLUMBIA

BRITISH COLUMBIA

Environmental Protection
10470 - 152 Street
Surrey, British Columbia
V3R 0Y3
Telephone: (604) 582-5200
Fax: (604) 584-9751

MINISTRY OF WATER,
LAND AND AIR PROTECTION

OPERATIONAL CERTIFICATE
ME-00030

Under the Provisions of the Waste Management Act and in accordance with the Greater Vancouver Regional District Liquid Waste Management Plan

Greater Vancouver Sewerage and Drainage District

4330 Kingsway

Burnaby, British Columbia V5H 4G8

shall operate a municipal waste water treatment plant located at 101 Bridge Road, West Vancouver, British Columbia, subject to the conditions listed below. Contravention of any of these conditions is a violation of the *Waste Management Act* and may result in prosecution. This operational certificate supercedes Waste Management Permit PE-00030.

1. AUTHORIZED DISCHARGE

1.1 This section applies to the discharge of effluent from **LION'S GATE WASTE WATER TREATMENT PLANT SERVING THE DISTRICT OF WEST VANCOUVER, DISTRICT OF NORTH VANCOUVER AND CITY OF NORTH VANCOUVER TO BURRARD INLET.** The site reference number for this discharge is E100923.

1.1.1 The maximum daily authorized rate of discharge is 318,000 cubic metres/day.

1.1.2 The maximum daily (flow proportioned 24-hour composite) concentration level of the discharge shall be:

5-day biochemical oxygen demand (BOD_5)	130. mg/L, maximum
Total suspended solids (nonfilterable residue) (TSS)	130. mg/L, maximum

R.H. Robb
Assistant Regional Waste Manager

Date Issued: February 17, 1959
Date Amended:
(most recent)

1.1.3 The maximum daily discharge loadings* for BOD_5 and TSS for the final effluent to be used for the calculation of annual operational certificate fees shall be:

*daily discharge loading is the total amount of contaminants discharged per day (contaminant concentration x rate of discharge)

Year	BOD_5 (tonnes/day)	TSS (tonnes/day)
2004	12.0	13.0
2005	12.5	13.5
2006	13.0	14.0
2007 and thereafter	13.5	14.5

1.1.4 The designated treatment works, approximately located as shown on attached Site Plan A, are:

- prechlorination facilities;
- influent screens;
- preaeration and grit removal facilities;
- primary sedimentation tanks;
- disinfection facilities; and
- a submerged outfall with diffuser extending a minimum of 228 metres offshore discharging to a minimum depth of 17.3 metres below mean low water.

1.1.5 The location of the facilities from which the discharge originates is Block D and E of District Lot 5521, Group 1, NWD.

1.1.6 The location of the point of discharge is the Burrard Inlet approximately 183 metres southeast of Block D and E of District Lot 5521, Group 1, NWD.

R.H. Robb
Assistant Regional Waste Manager

2. GENERAL REQUIREMENTS

2.1 Maintenance of Works

The operational certificate holder shall inspect the authorized works regularly and maintain them in good working order. Notify the Regional Waste Manager of any malfunction of these works.

2.2 Bypasses

The discharge of effluent which has bypassed the designated treatment works is prohibited unless the approval of the Regional Waste Manager is obtained and confirmed in writing.

2.3 Emergency Procedures

In the event of an emergency which prevents compliance with a requirement of this operational certificate, that requirement will be suspended for such time as the emergency continues or until otherwise directed by the Regional Waste Manager provided that:

a. Due diligence was exercised in relation to the process, operation or event which caused the emergency and that the emergency occurred notwithstanding this exercise of due diligence;

b. The manager is immediately notified of the emergency; and

c. It can be demonstrated that everything possible is being done to restore compliance in the shortest possible time.

Notwithstanding (a), (b), and (c) above, the manager may require the operation to be suspended or production levels to be reduced to protect the environment while the situation is corrected.

2.4 Process Modifications

The Regional Waste Manager shall be notified prior to implementing changes to any process that may adversely affect the quality and/or quantity of the discharge.

R.H. Robb
Assistant Regional Waste Manager

2.5 Disinfection

The effluent shall be disinfected between May 1 and September 30 so that the Burrard Inlet fecal coliform water quality objective is not exceeded at the edge of the initial dilution zone as described in the *Municipal Sewage Regulation*.

If chlorine is used, the effluent shall be dechlorinated prior to discharge to reduce the chlorine residual below the detection limit.

2.6 Facility Classification and Operator Certification

The operational certificate holder shall have the works authorized by this permit classified by the Environmental Operators Certification Program Society (Society). The works shall be operated and maintained by persons certified within and according to the program provided by the Society. Certifications must be completed to the satisfaction of the Regional Waste Manager. In addition, the manager shall be notified of the classification level of the facility and certification level of the operator with the highest certification level of the facility and changes of the operator and/or operator certification level of the operator with the highest certification level of the facility.

Alternatively, the works authorized by this operational certificate shall be operated and maintained by persons who the operational certificate holder can demonstrate, to the satisfaction of the Director, are qualified in the safe and proper operation of the facilities for the protection of the environment.

2.7 Sludge Wasting and Disposal or Utilization

Efforts should be taken to beneficially utilize the sludge wasted from the treatment plant. Utilization or disposal of the sludge shall be done in a manner satisfactory to the Regional Waste Manager, or as authorized by regulation under the *Waste Management Act*.

2.8 Posting of Outfall

A sign shall be erected along the alignment of the outfall above the high water mark. The sign shall identify the nature of the works. The wording and size of the sign shall be acceptable to the Regional Waste Manager.

R.H. Robb
Assistant Regional Waste Manager

3. MONITORING AND REPORTING REQUIREMENTS

3.1 Discharge Monitoring

3.1.1 Sampling and Analyses

Suitable sampling facilities shall be installed and maintained and composite or grab samples of the effluent authorized by section 1.1 shall be obtained for analyses as indicated below. A composite sample is to consist of a sample composited in proportion to flow over a 24 hour period (or approved flow proportional continuous sampler may be used). Proper care should be taken in sampling, storing and transporting the samples to adequately control temperature and avoid contamination, breakage, etc.

Parameter	Frequency	Sampling Type	Required Detection Limit
Chlorine residual*, mg/L	daily	grab	0.1 mg/L
TSS, mg/L	5 times/week	composite	5 mg/L
BOD_5**, mg/L	5 times/week	composite	10 mg/L
Fecal coliform*, MPN/100 mL	once/week	grab	20 MPN/100 mL
Ammonia, nitrogen, mg/L	2 times/month	grab	1.0 mg/L
Fish bioassay (rainbow trout), 96 hour LC50, %	monthly	grab	95% confidence limits

*between May 1 and September 30 only
**COD may be used in place of BOD_5 if BOD_5 is examined with COD every fifth sample.

3.1.2 Additional Sampling Parameters

By June 30, 2004, the operational certificate holder will submit to the Regional Waste Manager for approval, a list of additional substances that will be monitored in the effluent and has been reviewed by the Environmental Monitoring Committee. The list shall include the substance name, sampling frequency, sample type and required detection limit. The list of substances shall be reviewed every five years in consultation with the Environmental Monitoring Committee and an updated list shall be submitted to the Regional Waste Manager for approval.

R.H. Robb
Assistant Regional Waste Manager

3.1.3 Toxicity Failures

If the monthly bioassay test fails, the operational certificate holder will conduct a Toxicity Identification Evaluation (TIE) study for the purpose of determining the probable cause of the failure. The results of the failed monthly bioassay test and TIE study will be submitted to the Regional Waste Manager by the end of the month following the month that the bioassay test failure occurred.

3.1.4 Flow Measurement

Provide and maintain a suitable flow measuring device and record once per day the effluent volume discharged over a 24-hour period.

3.2 Monitoring Procedures

3.2.1 Sampling Procedures

Sampling is to be carried out in accordance with procedures described in the latest version of "British Columbia Field Sampling Manual for Continuous Monitoring plus the Collection of Air, Air-Emission, Water, Wastewater, Soil, Sediment, and Biological Samples, 2003 Edition (Permittee)," or by suitable alternative procedures as authorized by the Regional Waste Manager.

A copy of the above manual may be purchased from Queen's Printer Publications Centre, P. O. Box 9452, Stn. Prov. Govt. Victoria, British Columbia, V8W 9V7 (1-800-663-6105 or (250) 387-4609). A copy of the manual is also available for inspection at all Environmental Protection offices.

3.2.2 Chemical Analyses

Analyses are to be carried out in accordance with procedures described in the latest version of "British Columbia Laboratory Methods Manual for the Analysis of Water, Wastewater, Sediment and Biological Materials and Discrete Ambient Air Sample (March 2003 Permittee Edition)", or by suitable alternative procedures as authorized by the Regional Waste Manager.

A copy of the above manual may be purchased from Queen's Printer Publications Centre, P. O. Box 9452, Stn. Prov. Govt. Victoria, British Columbia, V8W 9V7 (1-800-663-6105 or (250) 387-4609). A copy of the manual is also available for inspection at all Environmental Protection offices.

R.H. Robb
Assistant Regional Waste Manager

3.2.3 Quality Assurance

All data analyses required to be submitted by the operational certificate holder shall be conducted by a laboratory acceptable to the Regional Waste Manager. At the request of the manager, the operational certificate holder shall provide the laboratory quality assurance data, associated field blanks, and duplicate analysis results along with the submission of data required under Section 3, of the operational certificate.

3.3 Outfall Inspection

The operational certificate holder shall have the outfall inspected once every five years by independent qualified personnel to ensure that it is in good condition. An inspection report shall be submitted to the Regional Waste Manager within 30 days after the inspection date. The first report shall be submitted by **January 31, 2006**.

3.4 Reporting

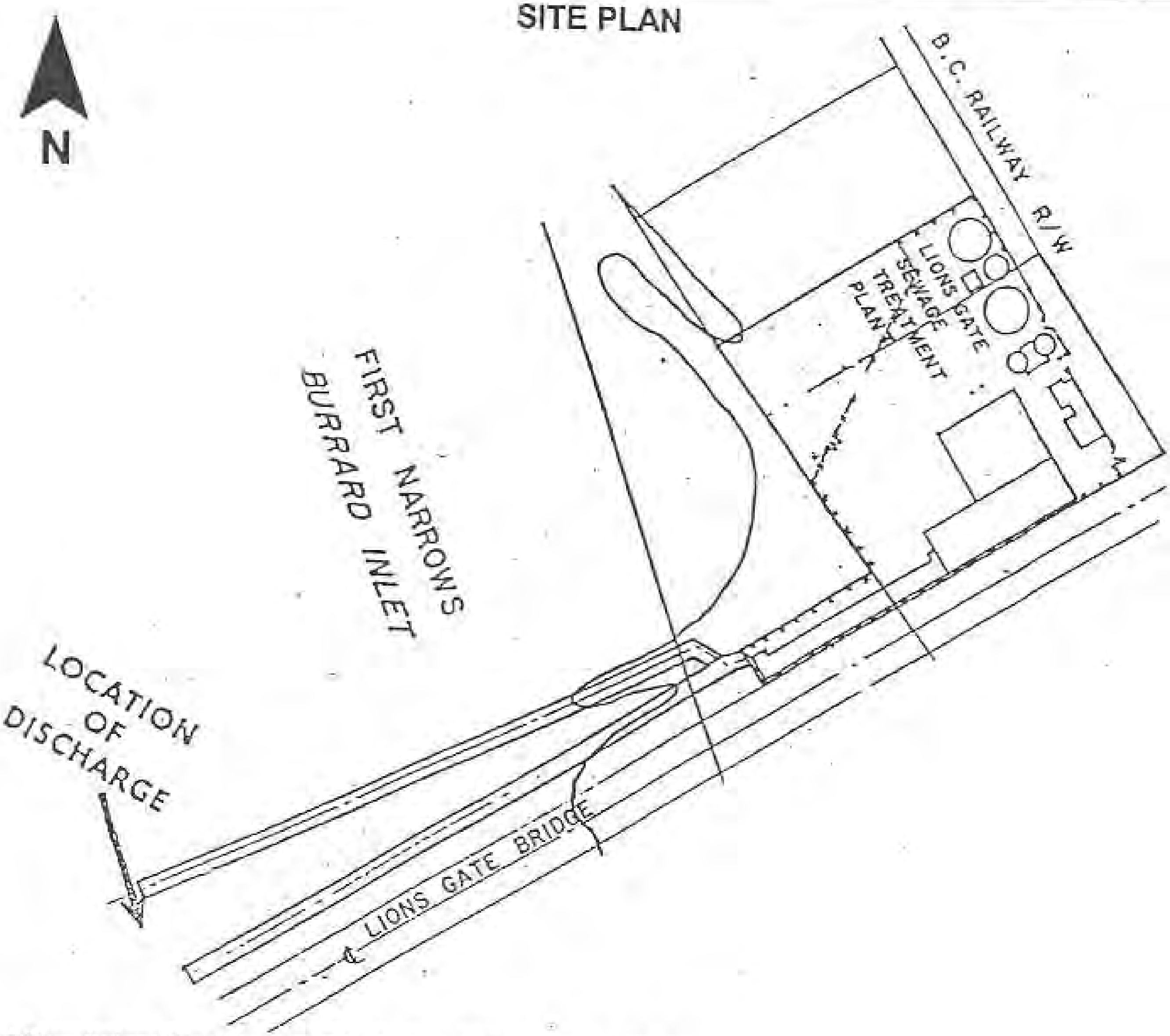
Maintain data of analyses and flow measurements, suitably tabulated, for inspection and post the data monthly on the operational certificate holder's Internet web site. Notify the Regional Waste Manager monthly of any data that is in noncompliance with requirements of this operational certificate.

The following information shall also be included in the liquid waste management plan biennial report:

- trends of the analytical data over the past two years and comparisons with data from the past years; and
- a summary of analytical data submitted in the monthly reports.

R.H. Robb
Assistant Regional Waste Manager

SITE PLAN



Legal Description: Block D & E, District Lot 5521, Group 1, NWD.

Location Map

Scale: 1:3600	
Site Plan: A	OC ME-00030
Date:	

R.H. Robb
Assistant Regional Waste Manager
Lower Mainland Region

APPENDIX D: LIFE CYCLE COST ESTIMATE DETAILS

IIWWTP Life Cycle Cost Estimate Details

Option	Interim 2021								Build-Out to Secondary 2036			
	Option 1 25% ADWF RTF		Option 2 50% ADWF RTF		Option 3 CEP Only		Option 4 CEP + 50% ADWF RTF no SCL		Option 1 TF/SC		Option 2 BAF	
Year	O/M	Capital	O/M	Capital	O/M	Capital	O/M	Capital	O/M	Capital	O/M	Capital
2004	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2005	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2006	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2007	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2008	$ -	$ 36,138,900	$ -	$ 67,313,763	$ -	$ 36,421,363	$ -	$ 53,712,544	$ -	$ -	$ -	$ -
2009	$ -	$ 34,093,302	$ -	$ 63,503,550	$ -	$ 34,359,777	$ -	$ 50,672,211	$ -	$ -	$ -	$ -
2010	$ 3,800,136		$ 5,077,711		$ 9,464,428		$ 9,620,051		$ -	$ -	$ -	$ -
2011	$ 3,585,034		$ 4,790,293		$ 8,928,705		$ 9,075,520		$ -	$ -	$ -	$ -
2012	$ 3,382,108		$ 4,519,145		$ 8,423,307		$ 8,561,811		$ -	$ -	$ -	$ -
2013	$ 3,190,668		$ 4,263,344		$ 7,946,516		$ 8,077,180		$ -	$ -	$ -	$ -
2014	$ 3,010,064		$ 4,022,023		$ 7,496,713		$ 7,619,982		$ -	$ -	$ -	$ -
2015	$ 2,839,683		$ 3,794,361		$ 7,072,371		$ 7,188,662		$ -	$ -	$ -	$ -
2016	$ 2,678,946		$ 3,579,586		$ 6,672,048		$ 6,781,756		$ -	$ -	$ -	$ -
2017	$ 2,527,308		$ 3,376,968		$ 6,294,385		$ 6,397,883		$ -	$ -	$ -	$ -
2018	$ 2,384,253		$ 3,185,819		$ 5,938,099		$ 6,035,739		$ -	$ 70,164,789	$ -	$ 63,689,253
2019	$ 2,249,295		$ 3,005,489		$ 5,601,980		$ 5,694,093		$ -	$ 66,193,197	$ -	$ 60,084,201
2020	$ 2,121,976		$ 2,835,367		$ 5,284,887		$ 5,371,786		$ -	$ 62,446,412	$ -	$ 56,683,208
2021									$ 5,154,959		$ 5,815,707	
2022									$ 4,863,168		$ 5,486,516	
2023									$ 4,587,895		$ 5,175,958	
2024									$ 4,328,203		$ 4,882,980	
2025									$ 4,083,210		$ 4,606,585	
2026									$ 3,852,085		$ 4,345,835	
2027									$ 3,634,042		$ 4,099,844	
2028									$ 3,428,342		$ 3,867,777	
2029									$ 3,234,285		$ 3,648,846	
2030									$ 3,051,212		$ 3,442,308	
2031									$ 2,878,502		$ 3,247,460	
2032									$ 2,715,568		$ 3,063,642	
2033									$ 2,561,856		$ 2,890,228	
2034									$ 2,416,846		$ 2,726,630	
2035									$ 2,280,043		$ 2,572,293	
2036									$ 2,150,984		$ 2,426,691	
2037									$ 2,029,230		$ 2,289,331	
2038									$ 1,914,368		$ 2,159,747	
2039									$ 1,806,008		$ 2,037,497	
2040									$ 1,703,781		$ 1,922,167	
2041									$ 1,607,340		$ 1,813,365	
2042									$ 1,516,359		$ 1,710,722	
2043									$ 1,430,527		$ 1,613,888	
2044									$ 1,349,554		$ 1,522,536	
2045									$ 1,273,164		$ 1,436,355	
2046									$ 1,201,098		$ 1,355,052	
2047									$ 1,133,112		$ 1,278,351	
2048									$ 1,068,973		$ 1,205,991	
2049									$ 1,008,465		$ 1,137,728	
2050									$ 951,382		$ 1,073,328	
2051									$ 897,531		$ 1,012,573	
2052									$ 846,727		$ 955,258	
2053									$ 798,799		$ 901,187	
2054									$ 753,584		$ 850,176	
2055									$ 710,928		$ 802,053	
2056									$ 670,687		$ 756,654	
2057									$ 632,724		$ 713,824	
2058									$ 596,909		$ 673,419	
2059									$ 563,122		$ 635,301	
2060									$ 531,247		$ 599,341	
Discounted Total O&M Cost	$ 31,769,471		$ 42,450,107		$ 79,123,440		$ 80,424,465		$ 82,216,819		$ 92,755,144	
Discounted Capital Costs		$ 70,232,201		$ 130,817,314		$ 70,781,140		$ 104,384,755		$ 198,804,399		$ 180,456,661
Total Capital and O & M Costs at Present Value	**$102,001,672**		**$173,267,420**		**$149,904,580**		**$184,809,220**		**$281,021,217**		**$273,211,805**	

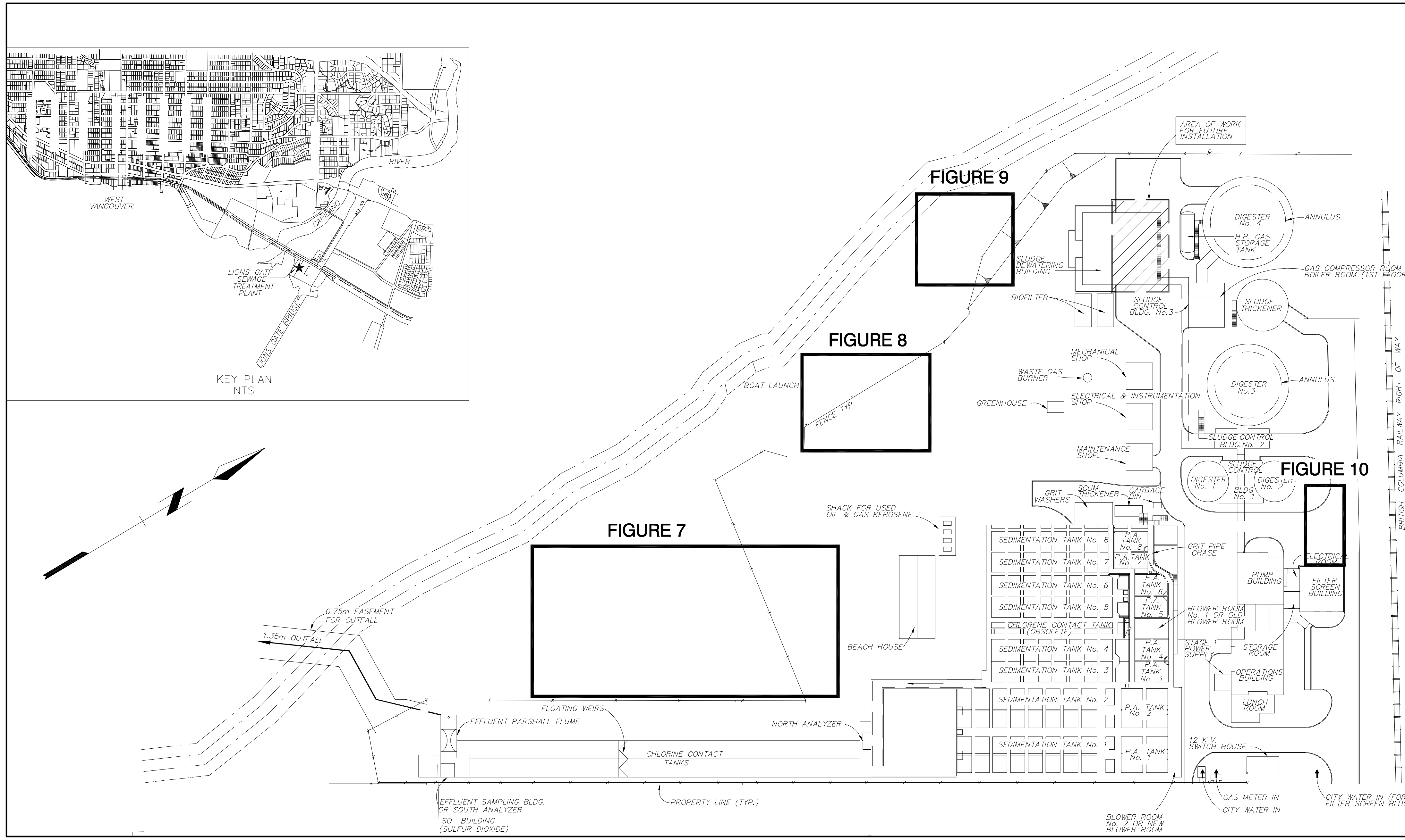
LGWWTP Life Cycle Cost Estimate Details

	Interim 2031						Build-Out to Secondary 2046	
Option	Option 1 CEP ONLY		Option 2A 50% BAF (No CEP)		Option 2B CEP+50% BAF		Option 3 2 x ADWF BAF	
Year	O/M	Capital	O/M	Capital	O/M	Capital	O/M	Capital
2004	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2005	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2006	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2007	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2008	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2009	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2010	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2011	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2012	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2013	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2014	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
2015	$ -	$ 6,399,971	$ -	$ 12,309,931	$ -	$ 14,057,027	$ -	$ -
2016	$ -	$ 6,037,709	$ -	$ 11,613,143	$ -	$ 13,261,346	$ -	$ -
2017	$ 1,329,114		$ 1,208,809		$ 1,905,433		$ -	$ -
2018	$ 1,253,882		$ 1,140,385		$ 1,797,578		$ -	$ -
2019	$ 1,182,907		$ 1,075,835		$ 1,695,828		$ -	$ -
2020	$ 1,115,950		$ 1,014,939		$ 1,599,838		$ -	$ -
2021	$ 1,052,783		$ 957,490		$ 1,509,281		$ -	$ -
2022	$ 993,192		$ 903,292		$ 1,423,850		$ -	$ -
2023	$ 936,973		$ 852,162		$ 1,343,255		$ -	$ -
2024	$ 883,937		$ 803,927		$ 1,267,221		$ -	$ -
2025	$ 833,903		$ 758,421		$ 1,195,492		$ -	$ -
2026	$ 786,701		$ 715,492		$ 1,127,823		$ -	$ -
2027	$ 742,171		$ 674,992		$ 1,063,984		$ -	$ -
2028	$ 700,161		$ 636,785		$ 1,003,758		$ -	$ 8,384,766
2029	$ 660,529		$ 600,741		$ 946,942		$ -	$ 7,910,157
2030	$ 623,141		$ 566,737		$ 893,341		$ -	$ 7,462,412
2031							$ 800,910	
2032							$ 755,575	
2033							$ 712,807	
2034							$ 672,459	
2035							$ 634,396	
2036							$ 598,486	
2037							$ 564,610	
2038							$ 532,651	
2039							$ 502,501	
2040							$ 474,057	
2041							$ 447,224	
2042							$ 421,909	
2043							$ 398,028	
2044							$ 375,498	
2045							$ 354,243	
2046							$ 334,192	
2047							$ 315,275	
2048							$ 297,429	
2049							$ 280,594	
2050							$ 264,711	
2051							$ 249,727	
2052							$ 235,592	
2053							$ 222,257	
2054							$ 209,676	
2055							$ 197,808	
2056							$ 186,611	
2057							$ 176,048	
2058							$ 166,083	
2059							$ 156,682	
2060							$ 147,813	
Total Discounted O & M Costs	**$ 13,095,343**		**$ 11,910,007**		**$ 18,773,623**		**$ 11,685,852**	
Discounted Capital Costs		**$ 12,437,680**		**$ 23,923,074**		**$ 27,318,373**		**$ 23,757,336**
Total Capital and O & M Costs at present value	**$25,534,000**		**$35,834,000**		**$46,092,000**		**$35,444,000**	

LIONS GATE WASTEWATER TREATMENT PLANT

Interim & Build-out to Secondary Stage
Preliminary Design Drawings

FIGURE No.	TITLE
1	TITLE SHEET
2	PROCESS SCHEMATIC
3	SUMMARY OF DESIGN CRITERIA
4	HYDRAULIC PROFILE
5	LAYOUT PLAN – CHEMICALLY ENCHANCED TREATMENT TILL 2031
6	LAYOUT PLAN – PARTIAL BIOLOGICAL TREATMENT TILL 2031
7	BIOLOGICAL AERATED FILTERS
8	DAF – PLAN & SECTION
9	DIGESTERS 5 AND 6 – PLAN & SECTION
10	CHEMICAL FEED RM & ALUM. STORAGE TANKS
11	LAYOUT PLAN – INTERCONNECTING PIPING

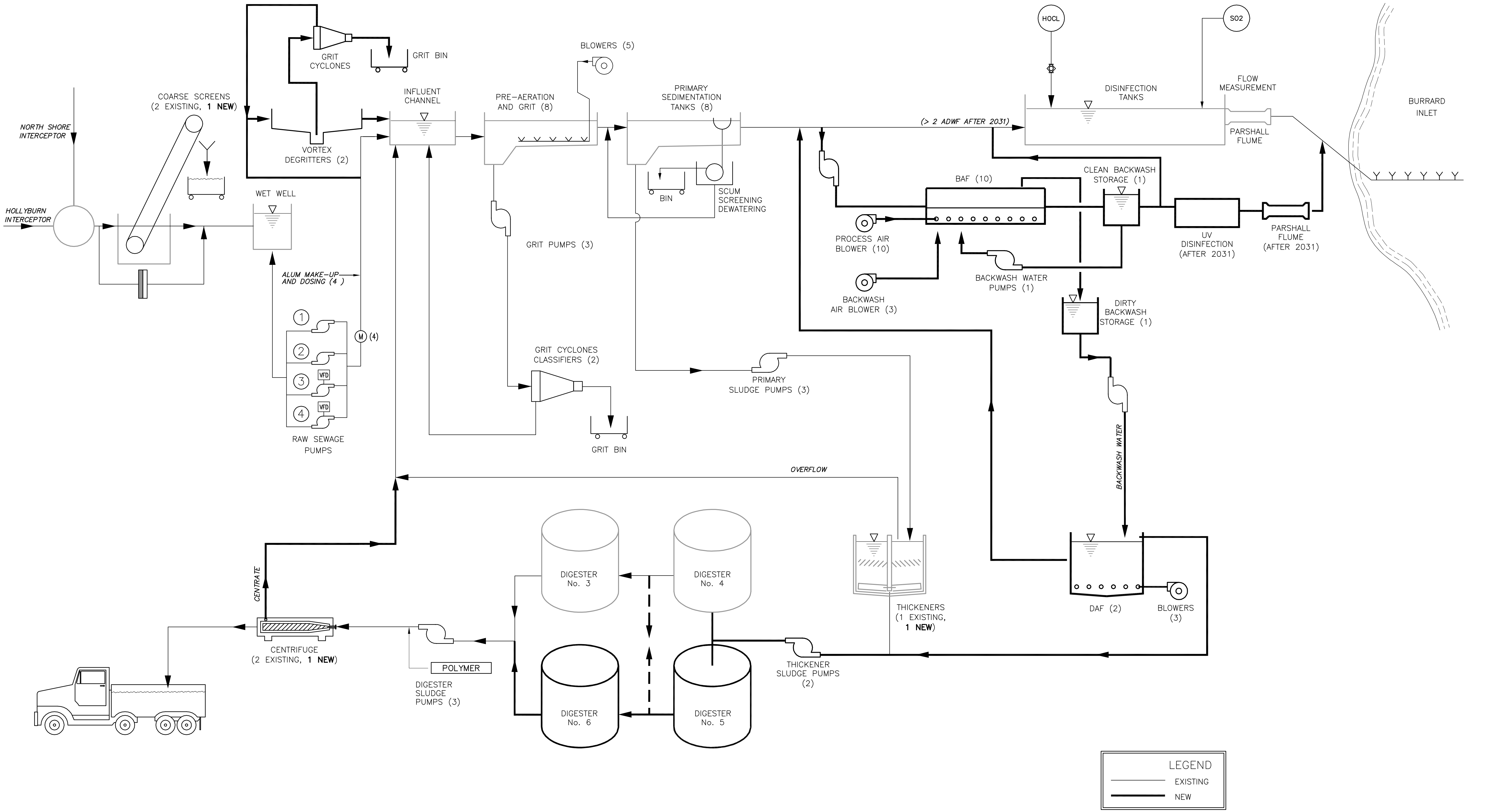


GREATER VANCOUVER REGIONAL DISTRICT

FACILITY PLANNING FOR IONA ISLAND AND LIONS GATE WASTEWATER TREATMENT PLANTS

PROJECT NUMBER: RFP 03-005

STANTEC CONSULTING LTD.
DAYTON & KNIGHT LTD.
117-00018 / 415.1.1



Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG2 1:1 05-01-07

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: I.R.-I
Drawn: JN
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

BUILD-OUT TO SECONDARY TREATMENT-2046

PROCESS SCHEMATIC

SCALE: AS SHOWN

DISTRICT FILE

FIGURE 2

DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

*SUMMARY OF DESIGN FLOWS AND LOADS

(DESIGN CASE SCENARIO)

	EXISTING (2003)	INTERIM UPGRADE (2031)	BUILD-OUT UPGRADE (2046)
FLOW			
ADWF (mL/d)	90	104	111
AAF (mL/d)	108	125	133
PWWF (mL/d)	304	337	356
LOAD			
MMBOD (t/d)	18	25	28
MMTSS (t/d)	22	28	32

*UNIT PROCESS DESIGN CRITERIA

1. PRIMARY SEDIMENTATION TANKS

- SURFACE OVERFLOW RATE : $50m^3/m^2/d$ (AVERAGE)
- SURFACE OVERFLOW RATE : $130m^3/m^2/d$ (PEAK)

	NO CEP	WITH CEP
DESIGN BOD REMOVAL (%)	27–30	55
DESIGN TSS REMOVAL (%)	60–64	80

2. BIOLOGICAL AERATED FILTER (BAF)

- SLUDGE YIELD : 0.94
- SLUDGE AGE : 2 days
- DESIGN BOD LOADING : $4.5kg/m^3/d$ (2031)
- DESIGN BOD LOADING : $5.5kg/m^3/d$ (2046)
- DESIGN TSS LOADING : $4.6kg/m^3/d$
- DESIGN HYDRAULIC LOADING : $144m^3/m^2/d$ (AVERAGE)
- DESIGN HYDRAULIC LOADING : $240m^3/m^2/d$ (PEAK)
- BACKWASH FLOW : 10% OF FLOW TREATED
- OXYGEN REQUIREMENT : 1.6kg O_2/kg BOD_5

3. GRAVITY THICKENER

- SOLIDS CONCENTRATION AFTER THICKENING : 5%
- DESIGN SOLIDS LOADING : $100kg/m^2/d$ (MAX. MONTH)

4. DISSOLVED AIR FLOTATION (DAF)

- SOLIDS CONCENTRATION AFTER DAF : 3.5%
- DESIGN SOLIDS LOADING : $48kg/m^2/d$ (WAS)

5. DIGESTER

- VSS LOADING : $2.2kg/d/m^3$
- SLUDGE VSS/TSS RATIO : 80% ~ 90%
- PRIMARY : 90%
- SECONDARY : 70%
- DESIGN HRT : 15d
- VS DESTRUCTION : 65%

6. DEWATERING – CENTRIFUGE

- CAPACITY : 900L/min
- SOLIDS CONCENTRATION AFTER DEWATERING : 35% PRIMARY SLUDGE
- SOLIDS CONCENTRATION AFTER DEWATERING : 27% SECONDARY SLUDGE

Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG3 1:1 05-01-07

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: ES
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

SUMMARY OF DESIGN CRITERIA

SCALE: AS SHOWN

DISTRICT FILE

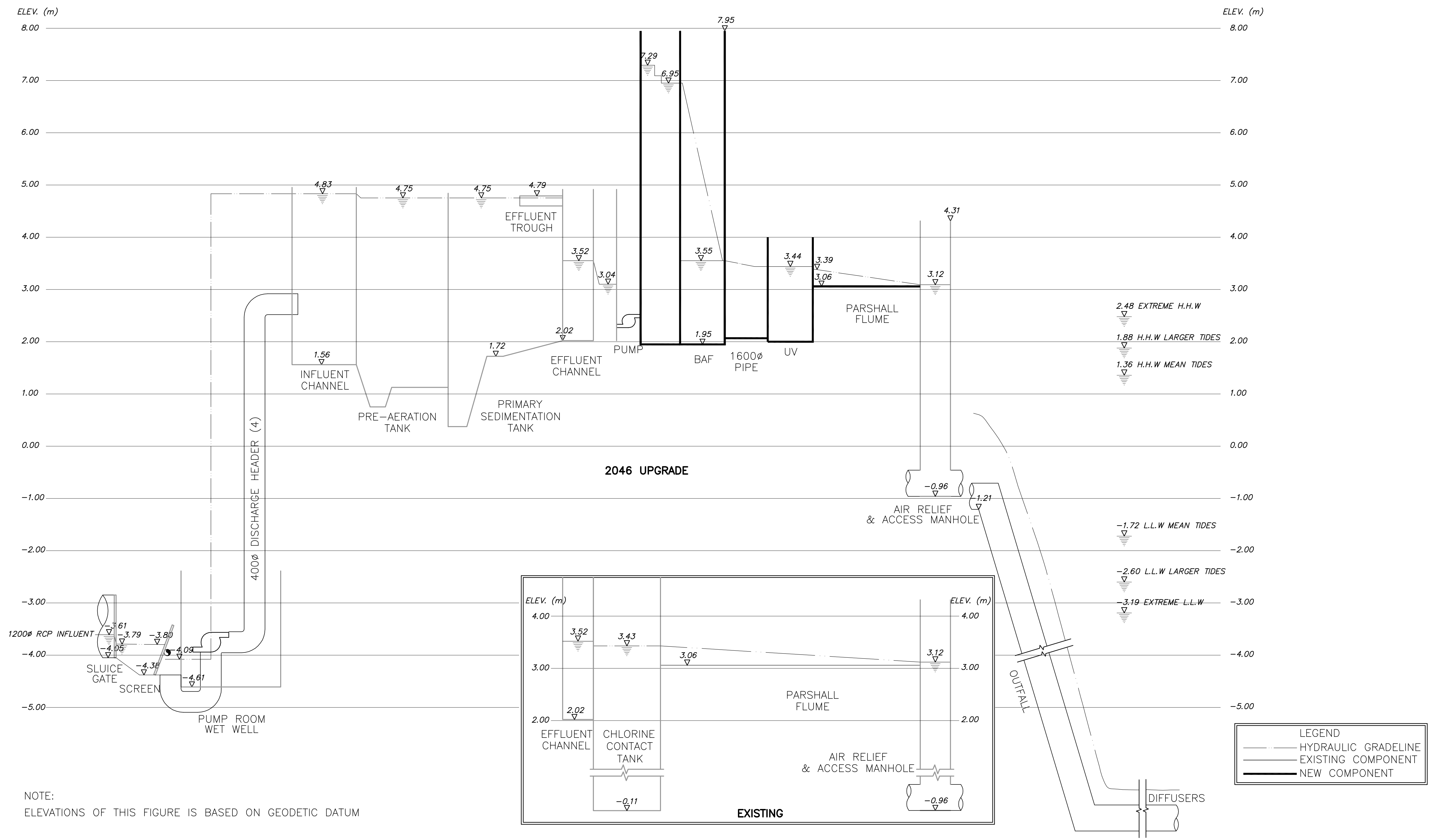
FIGURE 3

DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:



NOTE:
ELEVATIONS OF THIS FIGURE IS BASED ON GEODETIC DATUM

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DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG4 1:1 04-07-02

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design: I.R.-I
Drawn: JN
Checked:
Approved
Manager

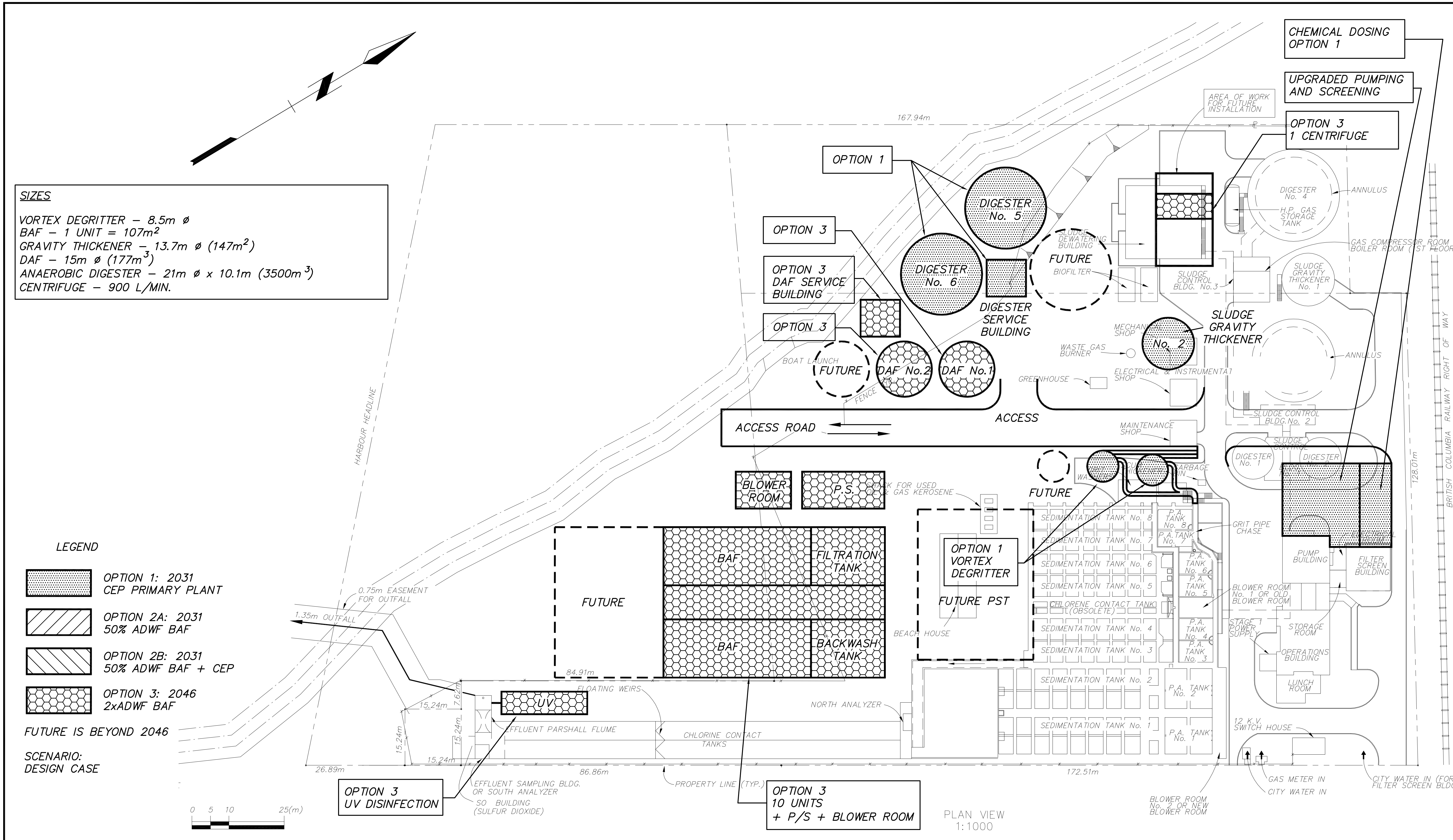
LIONS GATE WASTEWATER TREATMENT PLANT

CONCEPT DESIGN DRAWING

HYDRAULIC PROFILE

SCALE: AS SHOWN
DISTRICT FILE
FIGURE 4
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO ➡



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GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

LIONS GATE WASTEWATER TREATMENT PLANT

LAYOUT PLAN

CHEMICALLY ENCHANCED TREATMENT TILL 2031

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

Design:
Drawn: PTL
Checked:
Approved
Manager

SCALE: AS SHOWN
DISTRICT FILE
FIGURE 5
DOCUMENT CODE

Professional Seal

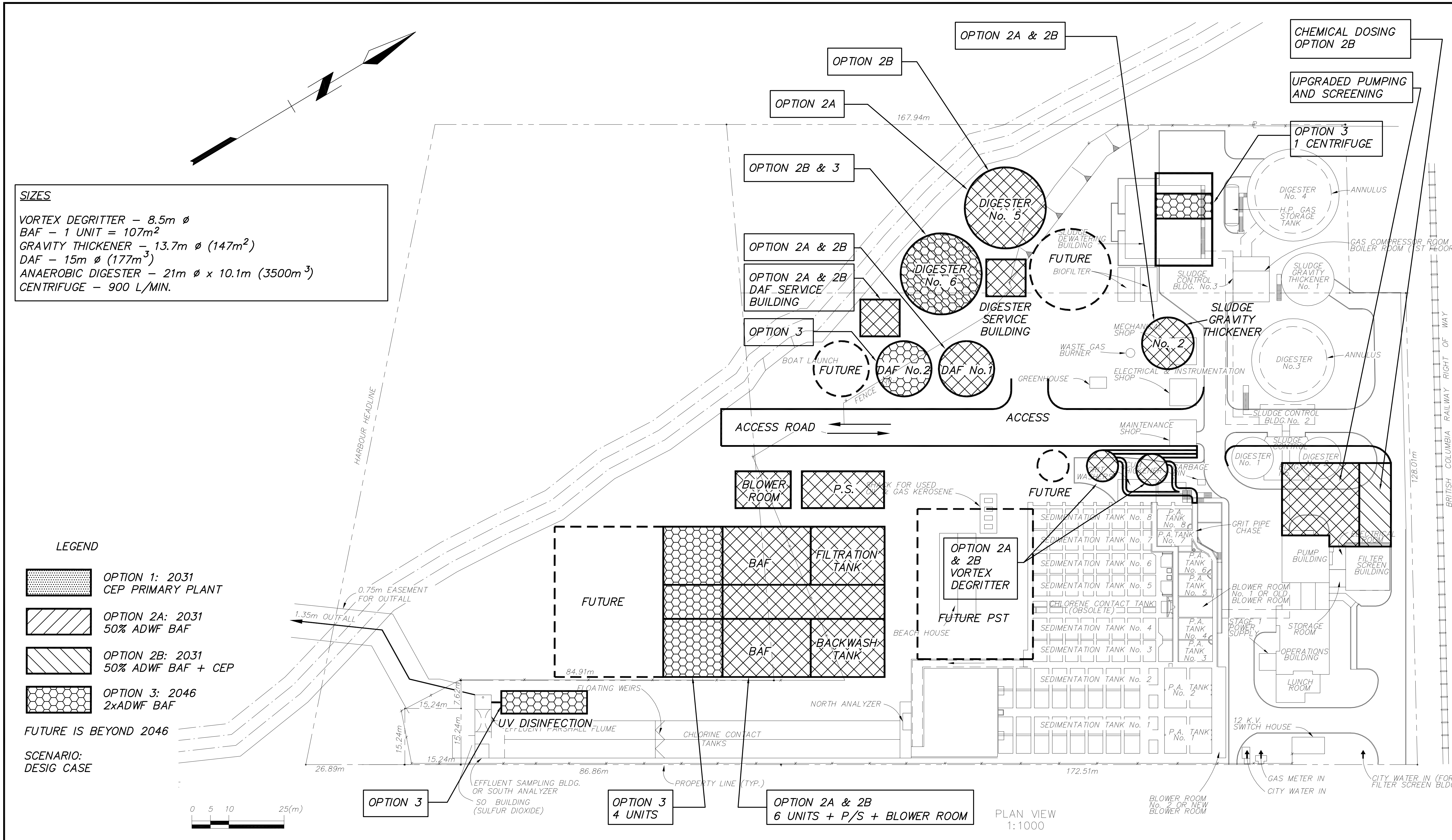
0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO →

XREF DWG:

SAROS DWG. ID No.:

DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG5 1:1 05-01-07



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Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

Professional Seal

Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: PTL
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

LAYOUT PLAN

PARTIAL BIOLOGICAL TREATMENT TILL 2031

SCALE: AS SHOWN

DISTRICT FILE

FIGURE 6

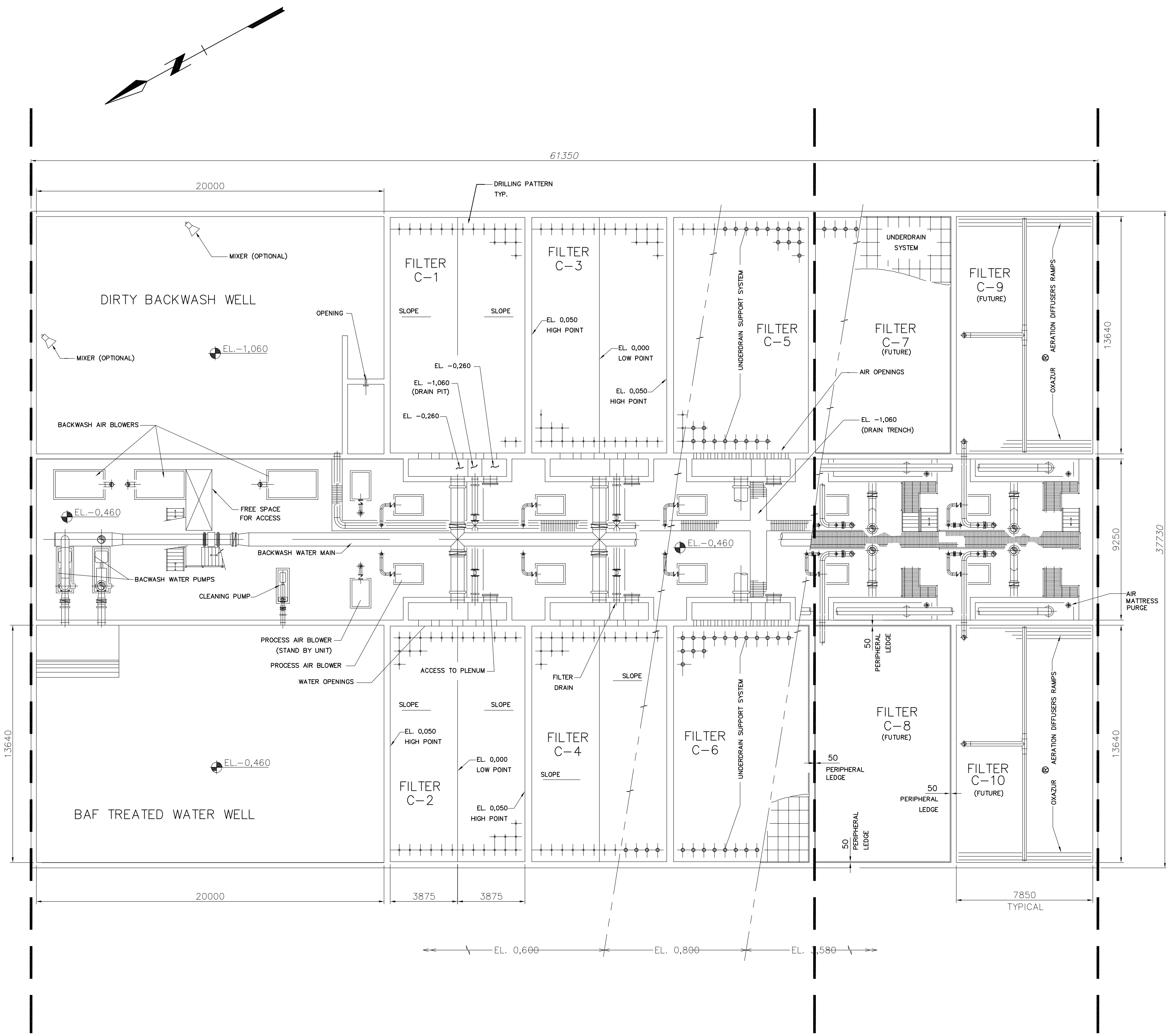
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

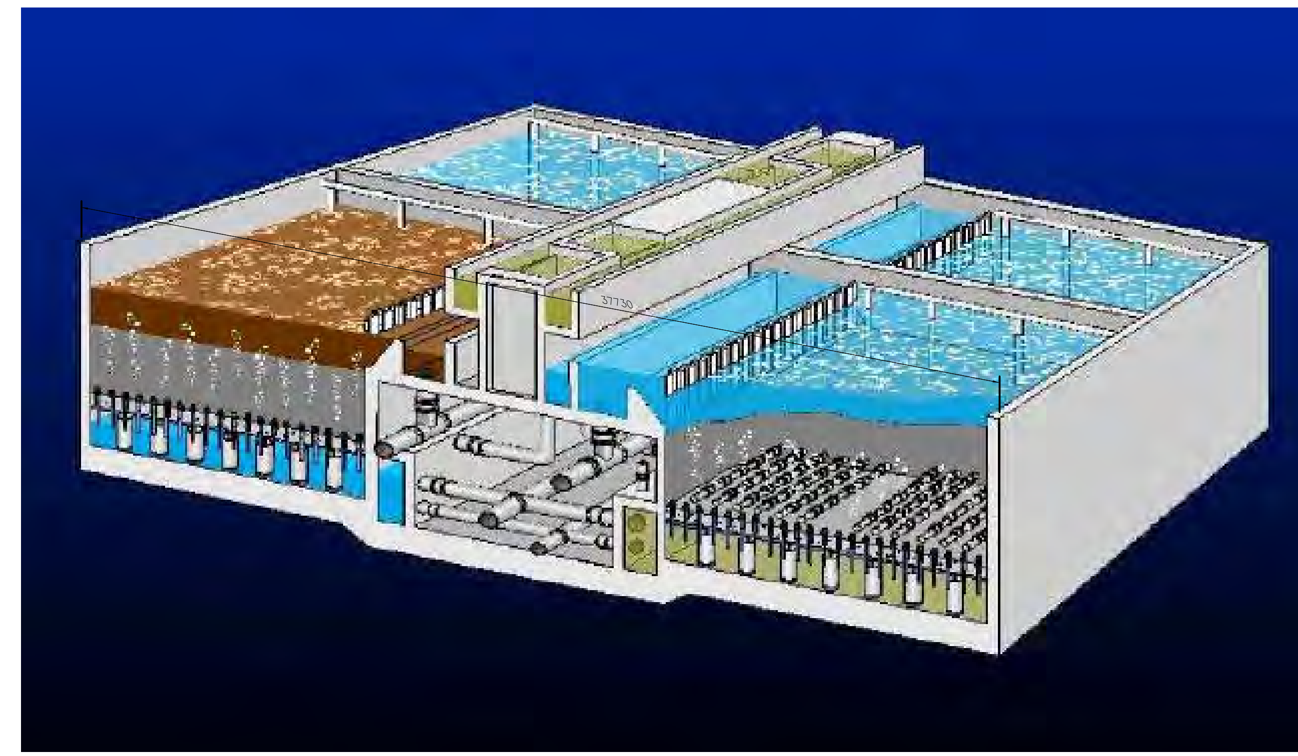
XREF DWG:

SAROS DWG. ID No.:

DWG. No. 415-1-112   ACAD DWG. 415-1-112-FIG6 1:1 05-01-07



PLAN VIEW
1:150



3D DRAWING OF TYPICAL BAF
N.T.S.

Dayton & Knight Ltd.

Stantec

DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG7 1:1 04-06-29

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

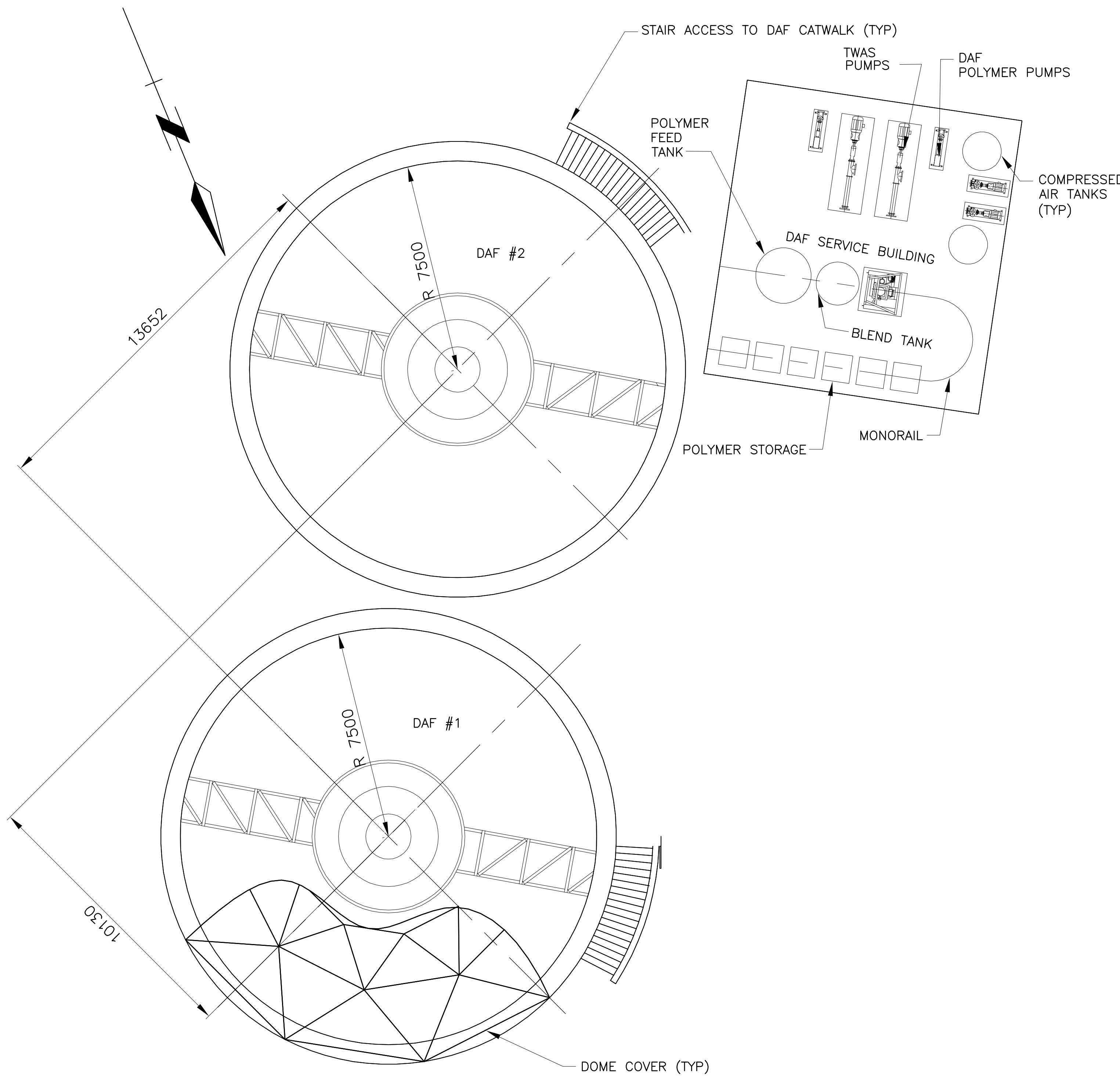
Design: I.R.-I
Drawn: KS
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

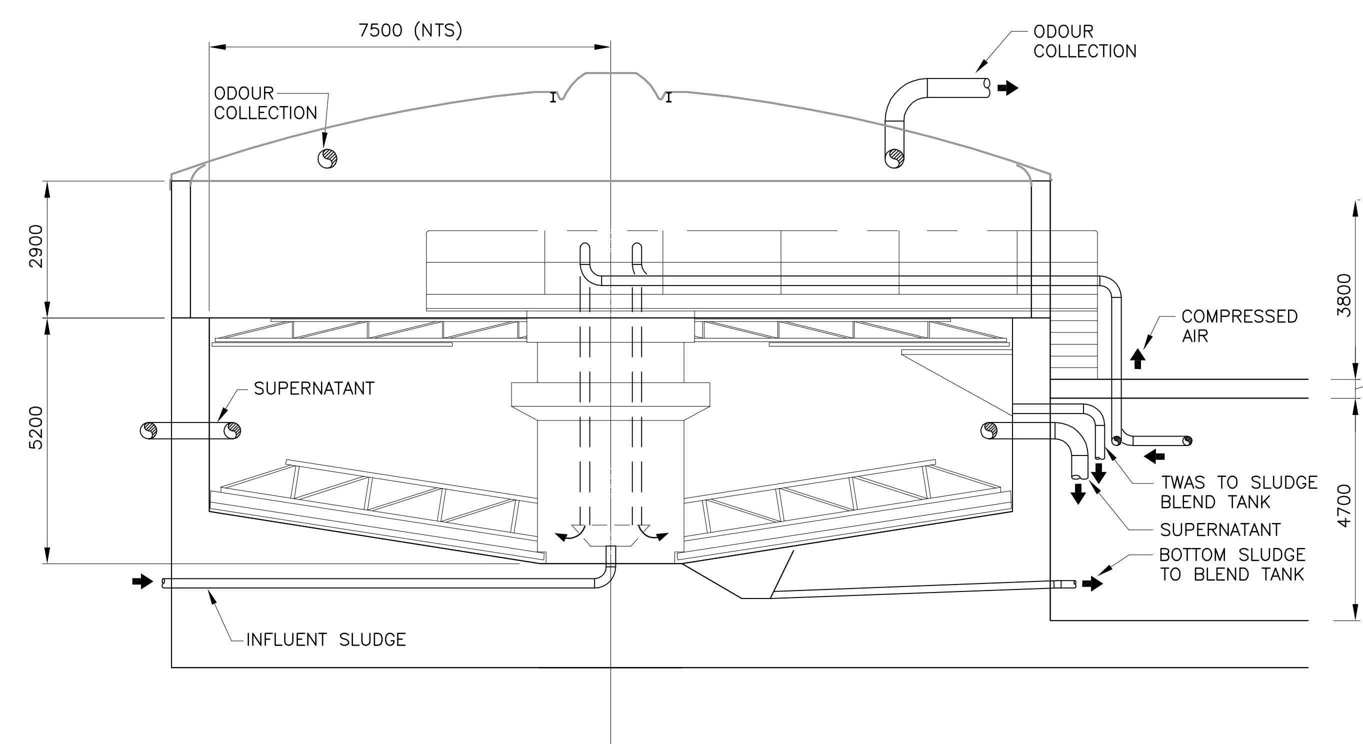
BIOLOGICAL AERATED FILTERS

SCALE: AS SHOWN
DISTRICT FILE
FIGURE 7
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO



PLAN – MAIN FLOOR
SCALE 1:200



SECTION
NTS

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DWG. No. 415–1–112 ACAD DWG. 415–1–112–FIG8 1:1 04–07–02

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description
P1	04/24/03	GKR	BMN			ISSUED FOR REVIEW

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

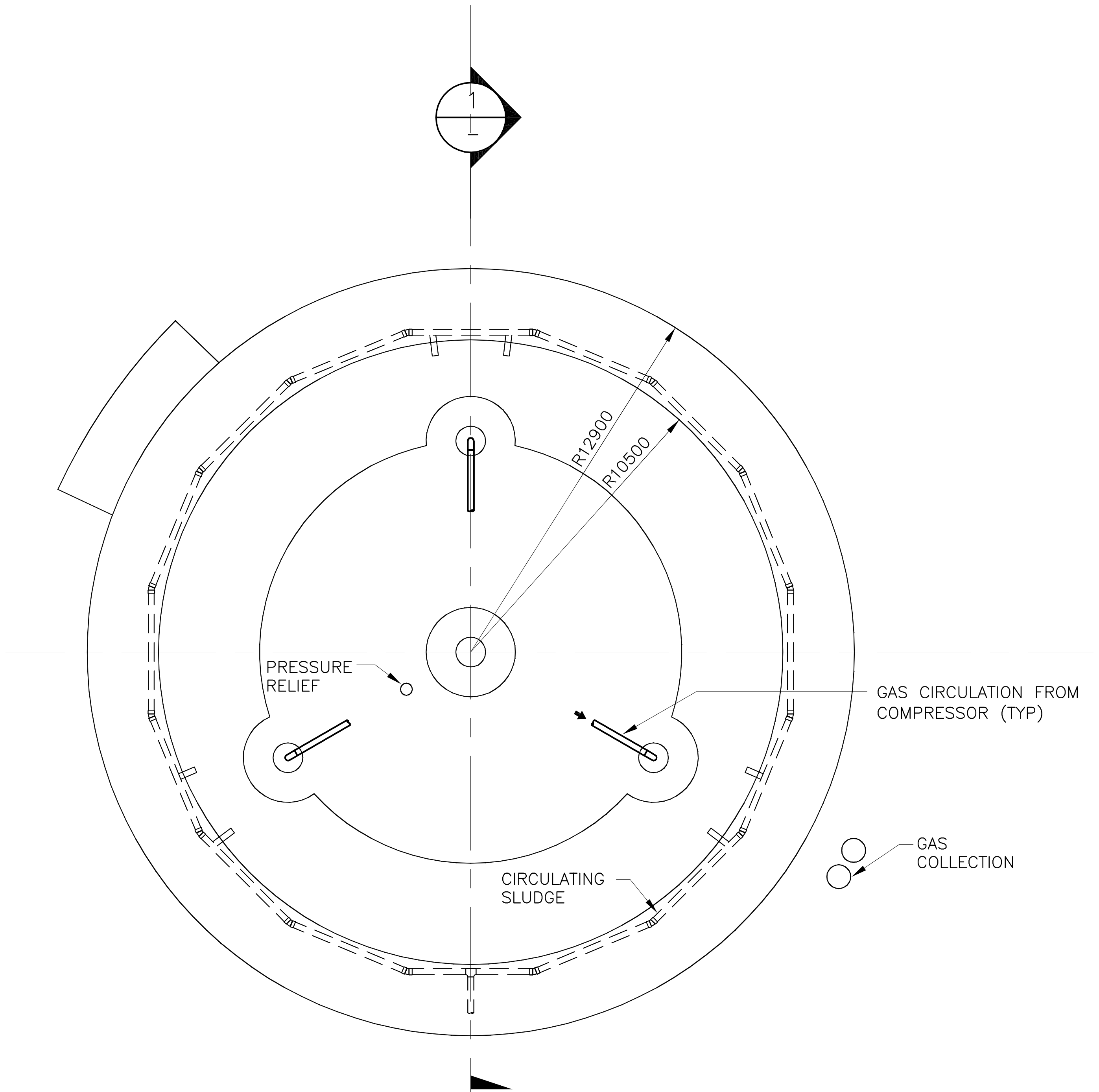
Design:
Drawn:
Checked:
Approved
Manager

GREATER VANCOUVER WATER DISTRICT
LIONS GATE WASTEWATER TREATMENT PLANT
DAF
PLAN & SECTION

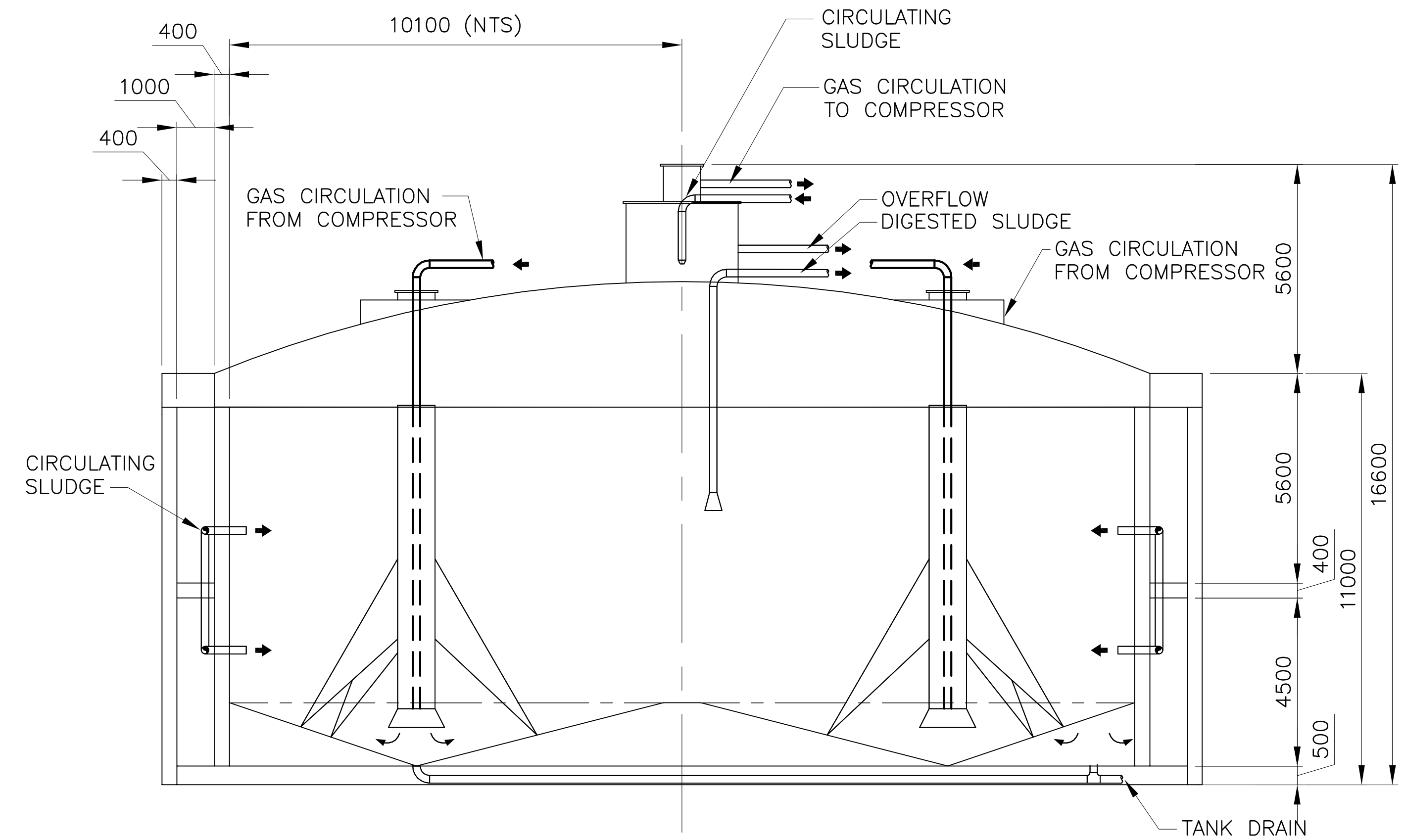
SCALE: AS NOTED
DISTRICT FILE 11700018
FIGURE 8
DOCUMENT CODE P01

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:
SAROS DWG. ID No.:



PLAN
1:200



SECTION
1:200

NOTE: PIPE SIZING NOT TO SCALE

Dayton & Knight Ltd.

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DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG9 1:1 04-07-02

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description
P1	–	–	–			ISSUED FOR REVIEW

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER WATER DISTRICT

Design:
Drawn:
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

DIGESTER 5 & 6

PLAN & SECTION

SCALE: 1:100

DISTRICT FILE 11700018

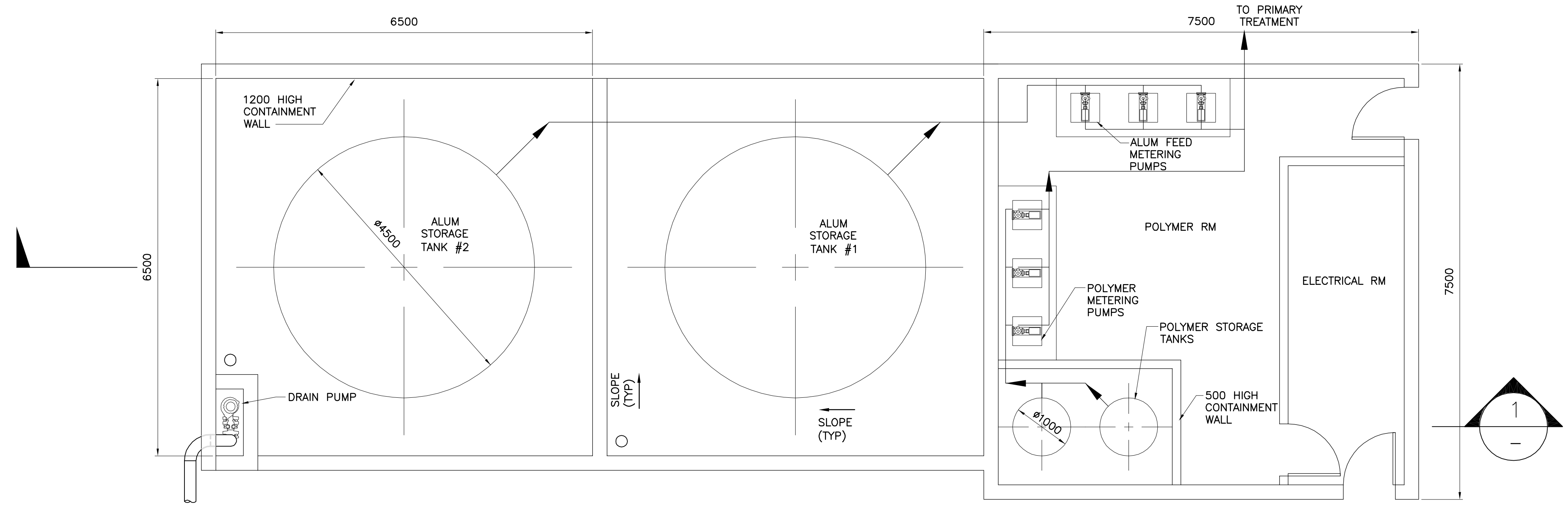
FIGURE 9

DOCUMENT CODE P01

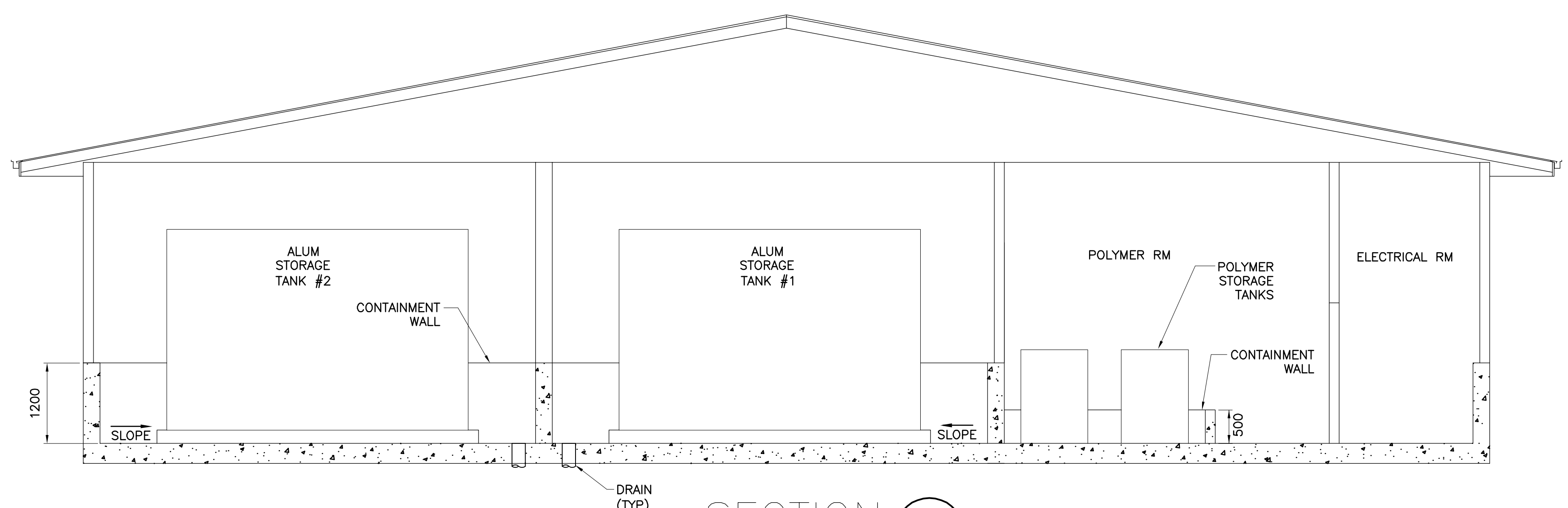
SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:

SAROS DWG. ID No.:



PLAN
1:100



SECTION 1 –
1:100

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DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG10 1:1 04-07-02

Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn:
Checked:
Approved
Manager

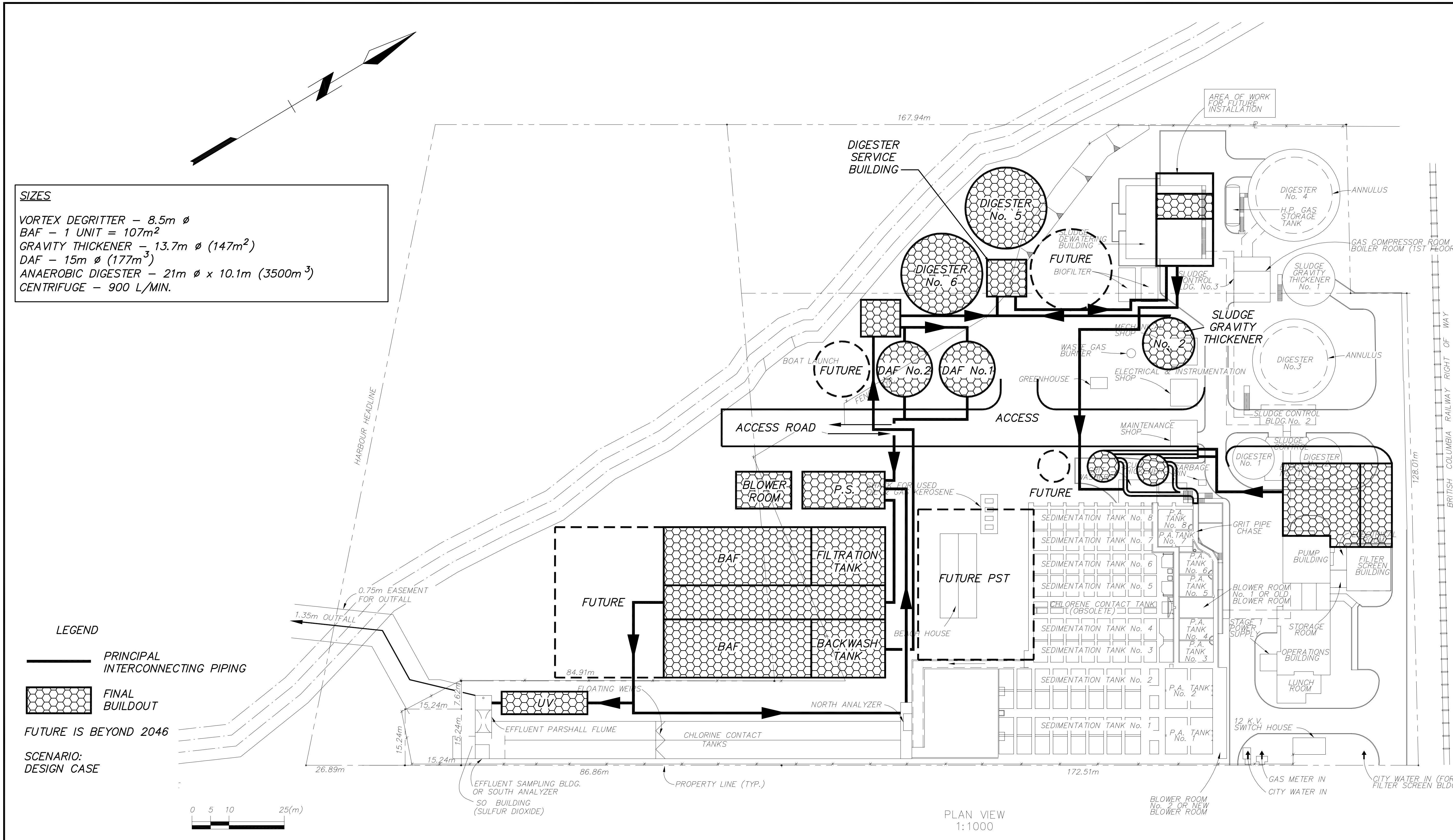
LIONS GATE WASTEWATER TREATMENT PLANT

CHEMICAL FEED RM & ALUM. STORAGE TANKS

SCALE: AS SHOWN
DISTRICT FILE
FIGURE 10
DOCUMENT CODE

SUPERSEDES PRINTS OF THIS DOCUMENT CODE WITH LETTERS PREVIOUS TO

XREF DWG:
SAROS DWG. ID No.:



Dayton & Knight Ltd.

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Professional Seal

Issue	Date	Des'n	Dr'n	Chkd	App'd	Description

0 20 Bar is 20mm On Original Drawing. If Not On This Sheet, Adjust Scales Accordingly.

GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT

Design:
Drawn: PTL
Checked:
Approved
Manager

LIONS GATE WASTEWATER TREATMENT PLANT

LAYOUT PLAN

INTERCONNECTING PIPING

SCALE: AS SHOWN

DISTRICT FILE

FIGURE 11

DOCUMENT CODE

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XREF DWG:

SAROS DWG. ID No.:

DWG. No. 415-1-112 ACAD DWG. 415-1-112-FIG11 1:1 05-01-07