Winnipeg

The City of Winnipeg Water & Waste Department

South End Water Pollution Centre (SEWPCC) Project Definition/Validation Report

FINAL PROJECT DEFINITION REPORT

MAY 2012

Submitted by:

Stantec

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South End Water Pollution Centre (SEWPCC) Project Definition/Validation Report

City of Winnipeg
Water and Waste Department

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Certificate of Authorization
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EXECUTIVE SUMMARY

INTRODUCTION

The Project Definition Report (PDR) is a compilation of the technical memorandum issued by Stantec and reviewed by the Program Team for the South End Water Pollution Control Center (SEWPCC) Project Definition / Validation Consulting Services project. The purpose of the SEWPCC project definition phase is to confirm the design criteria, validate that the process proposed by the Program Team has the capability to meet the design criteria, and identify any information gaps that must be addressed in the subsequent design phases.

The information gained from this assignment will be applied to:

- Confirm the feasibility of BAF in combination with a chemical based (Chem-P) removal system in terms of achieving compliance with effluent criteria specified by the Program Team; and,
- Define the scope of work for later stages in design.

In addition, Stantec was asked to update the Hauled Liquid Waste (HLW) business case with current information to determine if the construction of a new HLW facility at the SEWPCC remains the preferred overall solution for the City.

HAULED LIQUID WASTE STRATEGY

The SEWPCC currently accepts hauled liquid waste (HLW) for treatment on site. With the closure of the acceptance facility at the West End Water Pollution Control Center (WEWPCC), loads were subsequently redirected to both the North End Water Pollution Control Center (NEWPCC) and SEWPCC for treatment. Additional technical and financial information has become available for analysis since the construction of a new HLW acceptance facility at the NEWPCC. The Program Team requested that an update of the business case and associated factors used in the decision-making process be conducted since the addition of a HLW facility at the SEWPCC is seen as a major construction, financial, and operations commitment by the Department.

The purpose of the update to the hauled liquid waste (HLW) strategy report was to reassess the 2007 business case for HLW received at the SEWPCC and NEWPCC based on new information. Factors leading to the review of the 2007 HLW business case include:

- A 2008 wastewater characterization study at the SEWPCC found that HLW negatively impacted the nitrification growth kinetics;

- Better and more accurate financial information associated with the construction of a HLW facility at the NEWPCC; and,
- Revised data is now available regarding the amount of HLW received at both the NEWPCC and SEWPCC since 2007 and the influence this data has on projected loads for the future.

The HLW Plan is based on a user pay philosophy, this new information associated with receiving and treating HLW has the potential to alter the HLW business case on which the 2007 Plan is based.

This 2012 report updates the City's hauled liquid waste strategy allowing for the improved evaluation of the SEWPCC upgrade and expansion project and a clearer understanding of works to be implemented as part of the overall upgrade and expansion. The work is based on the 2007 report completed by Stantec titled "Winnipeg Regional Hauled Wastewater Plan". The evaluation includes completing analyses of technical, financial, and risk considerations to compare the option of continuing to receiving hauled liquid waste at both the SEWPCC and NEWPCC (referred to as Option 1 of the 2007 study) verse discontinuing receiving at the SEWPCC and only receiving at the NEWPCC (referred to as Option 6 of the 2007 study).

The key findings of the current technical analysis are as follows:

- The hauled waste annual average volumes have increased from 2005 through years 2008-2010 by less than 10% (from 300 m^3/d to 325 m^3/d).
- Using a design HLW flow of 367 m^3/d as a basis for evaluating alternatives in the 2007 study would appear to be reasonable given the marginal increase in flows from 2005 to 2010 (i.e., less than 10% increase).
- The original assumption that WEWPCC HLW flows would split 50/50 between SEWPCC and NEWPCC upon WEWPCC closure appears to be accurate. The resulting estimate of a 36/64 total HLW flow split between SE/NE plants in the Master Plan year of 2031 is also validated by 2008-2010 data that confirms a 34/66 flow split.

The key findings of wastewater characterization analysis are as follows:

- The impact on nitrifier growth associated with treating hauled waste within the main treatment process is expected to be greater at the SEWPCC than at the NEWPCC. Since the NEWPCC receives about three times the wastewater flow received at the SEWPCC, potential inhibition on nitrifiers from HLW would be less at the NEWPCC. Reduction in potential inhibition of nitrifiers is due to the greater protection afforded by dilution of toxic substances in the HLW that could negatively impact ammonia oxidizing bacteria growth. As such, it is expected that less mitigative measures would be required at the NEWPCC to protect the biological processes. A more robust treatment condition at the NEWPCC favors Option 6.Leachate typically has not been accepted at the SEWPCC for treatment in the past due to possible upset of the biological processes. Based on the current information associated with impaired nitrifier growth, and the additional measures and cost to accept and treat leachate, the program team provided Stantec with the direction that leachate will not be accepted at the SEWPCC in the future and to proceed with plant process designs with this understanding.

The key findings of the current financial analysis are as follows:

- The capital and O&M costs associated with implementing Option 6 (only receive HLW at the NEWPCC) are lower than Option 1 (continue receiving HLW at both the SEWPCC and NEWPCC). Capital cost estimates are $7.4M and $11.8M, and 25 year net present value of O&M costs are estimated at $11.6M and $18.7M for Options 6 and 1, respectively.

- The City will need to charge the haulers more to recover the higher capital, operation and maintenance costs associated with Option 6 and 1. The increase over the current basic fee of $17.07 charged to the haulers is estimated at $43 and $69 per typical 6.8 m^3 load, respectively.

- Trucking costs will be similar for Options 6 and 1 estimated at $14 and $13 per 6.8 m^3 load, respectively. Combined, these two costs will increase the total costs borne by hauled waste generators to $74 and $99 per typical 6.8 m^3 load for Option 6 and 1, respectively.

The key recommendations arising from the analysis of these important considerations include:

- Develop a plan to implement Option 6. The cost to the City and hauled waste generators is less to receive hauled waste at the NEWPCC only. This will result in lower hauler charge increases and lower cost increases for users dependent on hauled waste removal services.

- Review findings with affected stakeholders – This includes consulting with haulers and generators to inform them of recent findings and the proposed plan.

EFFLUENT CRITERIA

The Program Team provided specific effluent limits and averaging periods, as listed in Table E.1 below. This criteria defines performance requirement that the proposed design must achieve to satisfy Regulatory compliance requirements.

Table E.1 – Effluent Compliance Criteria

Parameter	Averaging Period	Limit	Units
Total Suspended Solids (TSS)	30-day rolling average	≤ 25.0[a]	mg/L
5-day Carbonaceous Biochemical Oxygen Demand ($cBOD_5$)	30-day rolling average	≤ 25.0[a]	mg/L
Total Phosphorus (TP)	30-day rolling average	≤ 1.0[a]	mg/L
Total Nitrogen (TN)	30-day rolling average	≤ 15.0[a]	mg/L
Ammonia Nitrogen - January	Daily never-to-exceed	≤ 1,975[a]	kg/day as N
Ammonia Nitrogen - February	Daily never-to-exceed	≤ 2,403[a]	kg/day as N
Ammonia Nitrogen - March	Daily never-to-exceed	≤ 4,196[a]	kg/day as N
Ammonia Nitrogen - April	Daily never-to-exceed	≤ 12,926[a]	kg/day as N
Ammonia Nitrogen - May	Daily never-to-exceed	≤ 5,311[a]	kg/day as N
Ammonia Nitrogen - June	Daily never-to-exceed	≤ 3,103[a]	kg/day as N
Ammonia Nitrogen - July	Daily never-to-exceed	≤ 1,517[a]	kg/day as N
Ammonia Nitrogen - August	Daily never-to-exceed	≤ 607[a]	kg/day as N
Ammonia Nitrogen - September	Daily never-to-exceed	≤ 703[a]	kg/day as N
Ammonia Nitrogen - October	Daily never-to-exceed	≤ 811[a]	kg/day as N
Ammonia Nitrogen - November	Daily never-to-exceed	≤ 1,152[a]	kg/day as N
Ammonia Nitrogen - December	Daily never-to-exceed	≤ 1,550[a]	kg/day as N
Ammonia Nitrogen - Year-round: Lethal to fish	Never-to-exceed	≤ 50%[b]	fish mortality
E-coli and Fecal coliform	30-day geometric mean	≤ 200.0	MPN/100 mL

**Notes: a – 24 hour effluent composite sample*
b – 96 hour static acute lethality test, pH adjusted

REGULATORY FRAMEWORK

The upgrades and expansion proposed for the SEWPCC are principally driven by regulator-defined effluent compliance requirements and projected population increases in the SEWPCC service area. Environment Act Licence No. 2716R, issued by Manitoba Conservation along with proposed Federal Wastewater Systems Effluent Regulations, stipulates the effluent limits and averaging periods that must be achieved to be in compliance. This section discusses the effluent criteria provided by the Program Team and future regulatory trends for target parameters along with their potential influence on design requirements. Key considerations are as follows:

- TSS and $cBOD_5$ limits for the proposed design are to be based on achieving less than 25 mg/L on a 30-day rolling average. In a draft Licence revision, Manitoba Conservation has proposed changing the averaging period to a “not-to-exceed 98% of the time” basis. This “not-to-exceed” 98% requirement will translate into significant additional costs. The added costs would be in the range of $60 million while providing only marginal additional environment benefit (City of Winnipeg, Administration Report, February 2011). This assignment has not addressed the treatment of requirements or the estimated cost to achieve compliance but believes the previous assessments are representative of costs and benefits.

- Ammonia nitrogen limits for the proposed design, as indicated in the Licence, is on a never-to-exceed daily basis that varies by month. The month-specific daily limit stipulated by Manitoba Conservation (for compliance) in turn establishes the design requirements for nitrification.

- TN and TP limits for the proposed design are to be less than 15 mg/L and 1mg/L, respectively, as measured by a 30 day rolling average. Trends in the North America regulatory environment indicate a move towards lower TN and TP limits if scientific information dictates that higher levels of treatment are required to protect the environment or in cases where water reclamation is required for potable water.

- Fecal coliform and E. coli limits for the proposed design are to be less than 200 MPN/100 mL as measured by a 30 day geometric mean and strongly influenced by untreated wet weather flow by-passes.

- The Licence as written requires that all flows that reach the SEWPCC be treated to achieve the stipulated limits regardless of the climatic conditions or river flood stages prevalent at any given time.

DESIGN FLOWS AND LOADS VALIDATION

The flows and loads for the design years 2031 and 2061 are based on the City's projected SEWPCC service populations of 270,000 and 400,000, respectively. Using actual SEWPCC influent flows, service population and a conservative winter infiltration rate of 45 liters per capita per day, the dry weather wastewater generation rate was determined to be 278 liters per day on a per capita basis. Historical data was used to develop a model to predict peak flows at the SEWPCC for the 2031 and 2061 design years. In developing the future peak flows it was recognized that the collection system currently limits the flows that can be conveyed to the SEWPCC. Annual average, maximum 30 day average, maximum 7 day average and maximum day flows have been determined for each season.

Design loading values were determined using actual raw wastewater data provided by the City from 2005 through 2011. Statistical analysis of the data was applied and the 98th percentile was selected for design purposes. Seasonal loadings were developed by establishing per capita loadings based on annual average loads and adjusting per capita loads to account for removal of the hauled liquid wastewater component and the year-round addition of flows from the Windsor Park sewer district.

A stress pattern representative of projected future 2031 and 2061 flows and loads at the SEWPCC was developed. The stress pattern assumes year round flows from Windsor Park and does not include hauled liquids waste at the SEWPCC. The stress pattern was used to assess the performance of the proposed upgrades with respect to achieving compliance with the criteria provided by the Program Team. The analysis found that compliance with ammonia effluent criteria was the parameter controlling the design requirements of the proposed upgrade.

BIOSOLIDS HANDLING AND TREATMENT

The Program Team is in the process of developing an overall biosolids handling plan for the City's three wastewater treatment plants. The results of the overall plan are not available for direction in this report. For the purpose of this phase of the project Stantec was directed to base the analysis of biosolids handling and treatment at the SEWPCC on anaerobic digestion followed by dewatering. The assumed process includes blending and thickening of sludge from the chemically enhanced primary treatment (CEPT), high rate clarification (HRC) and backwash clarification processes. The thickened sludge will be digested using two stage anaerobic digestion and dewatered using centrifuges. A preliminary mass balance was completed to determine the impact of solids handling and treatment on the liquid stream processes.

Filtrate from the thickening process and centrate from the dewatering step will need to undergo side stream treatment prior to discharge back to the main stream process. While the expected reject water from filtrate and centrate are expected to be in the 1 ML/d range, the recycled nitrogen (especially ammonia) and phosphorus load can be in the range of 20 to 25% of the total plant load. Previous engineering analyses found treatment of this side stream to be cost-effective at the NEWPCC. As such, centrate treatment with the same performance as that currently provided at the NEWPCC was assumed in the mass balance analyses.

CONCEPTUAL DEVELOPMENT

The development of the concept for the SEWPCC upgrades includes three varying levels of treatment based on the influent flow to the plant. Flows up to 200 ML/d receive screening, grit removal, primary treatment, secondary treatment and disinfection. Flows greater than the 200 ML/d but less than (the max day) 325 ML/d receive screening, grit removal, primary treatment and disinfection, while flows greater than 325 ML/d only receive screening and grit removal. The concept includes the following process components:

- A Headworks facility consisting of an upgraded raw sewage pumping station (420 ML/d), conversion of existing 12 mm climber bar screens to 6 mm perforated plate screens (420 ML/d) and expansion of the existing aerated grit removal system (de-rated to 200 ML/d) with two new vortex grit removal units (220 MLD).

- The existing primary clarifiers would be de-rated to 150 ML/d and will be operated as chemically enhanced primary treatment (CEPT) to improve performance and precipitate phosphorous.

- Two (2) HRCs are required to provide primary treatment for wet weather flows greater 150 ML/d and less than 325 ML/d. Solids removed from the HRCs will be pumped to the solids handling facility.

- An intermediate pump station will be constructed to pump the primary effluent (150 ML/d from existing primary clarifiers and 50 MLD from the new HRC), backwash clarified effluent (maximum of 18 ML/d).

- A new BAF is required for carbon removal and nitrification / denitrification for flows up to 200 ML/d. Backwash waste generated by the process is pumped to backwash clarifiers to settle the solids before the clarified effluent is returned to the BAF via the intermediate pump station. Two (2) of the existing 33.5 m diameter secondary clarifiers will be converted to backwash clarifiers. Settled solids will be pumped to the solids handling facility for thickening.

- The design of the upgrade and expansion for the SEWPCC has provisioned for the possible conversion of the main stream treatment processes from a chemical-based phosphorus removal system to a biological-based phosphorus removal system. As such, due consideration will be given to provide the flexibility to return the final clarifiers to their original function and operations if the decision is made to remove phosphorus by biological means in the future.

- Disinfection is provided for all flows up to 325 ML/d. A new UV disinfection facility is proposed for the secondary effluent from the BAF. The facility will be constructed adjacent to the existing UV facility. Flow greater than the capacity of the BAF treatment process up to 325 ML/d is proposed to be disinfected by a chlorination and dechlorination system. The proposed chlorination and dechlorination system is feasible for disinfection of the primary effluent from the HRC in conjunction with the reuse of two of the existing HPO reactors. .

- A new solids handling facility is proposed to stabilize and dewater solids generated by the CEPT, HRC and backwash clarifiers. The solids handling facility includes blending, thickening (drum thickeners), 2 stage digestion, sludge storage, dewatering (centrifuges) and centrate treatment (SBR).

The process components have been arranged on the SEWPCC site and space has been allocated for future growth (to 2061), potential future changes to the effluent requirements (TN ≤ 10 mg/L, TP ≤ 0.3 mg/L) and potential future conversion to biological phosphorous removal. A preliminary hydraulic profile for the 2031 concept has been developed to provide disinfection for flows up to the peak day flow (325 ML/d) and to protect against flooding at the total pumped flow (420 ML/d) for river levels greater than the 50 year return period (elevation 230.00 m). Since the existing collection system limits the flow that can be safely conveyed to the SEWPCC, it has been assumed that upgrading of the hydraulic capacity of the collection system will be undertaken in the future as needed to facilitate unconstrained growth in the south end services area. The hydraulic design requirements of the SEWPCC for projected 2061 flows are a key consideration in the staged expansion of the plant beyond 2031. Accordingly, hydraulic requirements for projected 2061 flows have been considered in the conceptual layouts for the plant upgrade and expansion and will be taken into consideration as part of the schematic design.

CIVIL ASSET CONDITION ASSESSMENT

Stantec conducted a General Facility Condition Assessment of the SEWPCC in the spring of 2008 to evaluate the current structural and building envelope conditions of the facility and to identify potential problem areas. The 2008 assessment included a general, non-destructive, walk-through review of the structural and building envelope systems only. Stantec has updated the list to reflect items that have been completed since the review in 2008.

Vector Corrosion Technologies was contracted in the spring of 2008 to perform testing on the concrete of Primary Clarifier #3 and Primary Clarifier #1. The investigation assessed the concrete condition and the results of the testing indicated that there is very little corrosion activity and no signs that the reinforcing steel is deteriorating.

General findings applicable to the entire facility include identification of hairline shrinkage cracks present in concrete slabs and walls throughout the facility (requires patching and monitoring), detection of mold is on the brick veneer at various locations, additional cleaning and application of a protective coating is required, and exterior caulking replacement at most windows. Areas where minor issues were noted include the administration building, the maintenance/service building, the standby generator building, Primary Clarifier No. 3, Oxygen Reactors Nos. 3 and 4, the PSA building, sludge thickening and the sludge truck bay and the UV building. Areas where more immediate repair work is required include the pump chamber and screen building, the grit building, Primary Clarifier Nos. 1 and 2, Oxygen Reactors Nos. 1 and 2 and Secondary Clarifiers Nos. 1, 2 and 3.

Additional investigation work is required at the concrete dry well wall of the raw water pump chamber and for all four oxygen reactors. This work should be scheduled immediately as it is important to continued plant operation and may impact the usability of the existing infrastructure for the new treatment system. No work is required at the odor control stack.

The interior condition of the high purity oxygen (HPO) tanks is unknown. Since the HPO tanks are intended to be reused for wet weather disinfection reasons, and possibly for biological phosphorus removal in the future, their interior condition needs to be known to determine and establish how much restorative work, if any, may be required before these tanks can be repurposed. Scheduling and detailing of a work plan needs to be coordinated with the Program Team as soon as possible to permit the work to be completed in a planned and orderly fashion within seasonal constraints.

RISK ANALYSIS AND MITIGATION STRATEGIES

A workshop was held with the Program Team and Stantec on December 5, 2011 to identify project risks related to the delivery of the preliminary design and to determine mitigation strategies. The key project drivers identified through the workshop were cost certainty, lowest whole life costs and schedule. Risks have been identified and ranked according to the likelihood and the severity of the consequence if the risk event were to occur. The outcome of the risk analysis workshop is expressed as a risk matrix working document that identifies the preliminary design risks, a risk prioritization framework, proposed mitigation strategies and risk ownership.

Table of Contents

EXECUTIVE SUMMARY E.1

1.0 INTRODUCTION ... 1.1
1.1 INTRODUCTION ... 1.1
1.2 PROJECT OBJECTIVES ... 1.2
1.3 REPORT DEVELOPMENT ... 1.4

2.0 HAULED LIQUID WASTE STRATEGY ... 2.1
2.1 PURPOSE ... 2.1
2.2 BACKGROUND ... 2.1
2.3 TECHNICAL AND FINANCIAL ANALYSES ... 2.2
2.3.1 Hauled Waste Flows ... 2.3
2.3.2 Financial Impacts ... 2.4
2.3.2.1 City Costs ... 2.4
2.3.2.2 Hauler Costs ... 2.7
2.3.2.3 Total Costs (City + Hauler) ... 2.8
2.4 RISK ASSESSMENT ... 2.9
2.5 CONCLUSIONS AND RECOMMENDATIONS ... 2.11

3.0 REGULATORY FRAMEWORK ... 3.1
3.1 BACKGROUND ... 3.1
3.2 REGULATORY FRAMEWORKS / ISSUES ... 3.1
3.3 CURRENT EFFLUENT CRITIERA ... 3.2
3.4 FUTURE REGULATORY TRENDS FOR TARGET PARAMETERS ... 3.3
3.4.1 $cBOD_5$ and TSS ... 3.3
3.4.2 Licence Limits for Ammonia-N ... 3.4
3.4.3 Total Nitrogen (TN) and Total Phosphorus (TP) ... 3.7

4.0 DESIGN FLOWS AND LOADS VALIDATION ... 4.1
4.1 INTRODUCTION ... 4.1
4.2 DEVELOPMENT OF DESIGN FLOWS ... 4.3
4.2.1 Flow Technical Analyses ... 4.3
4.2.2 Population Projections ... 4.5
4.2.3 Dry Weather Per Capita Wastewater Generation Rate ... 4.6
4.2.4 Peak Flows to the SEWPCC ... 4.8
4.3 DEVELOPMENT OF DESIGN LOADINGS ... 4.16
4.3.1 Loading Technical Analysis ... 4.16

5.0 BIOSOLIDS HANDLING AND TREATMENT ... 5.1
5.1 BACKGROUND ... 5.1
5.2 PROCESS DESCRIPTION ... 5.1
5.2.1 Raw Wastewater Pumping and Headworks ... 5.2

Table of Contents

5.2.2 High Rate Clarification (HRC) 5.2
5.2.3 Chemically Enhanced Primary Treatment (CEPT) 5.3
5.2.4 Biologically Active Filtration (BAF) 5.3
5.2.5 Solids Handling Processes 5.5

6.0 OPERATIONAL PHILOSOPHY 6.1
6.1 TECHNICAL APPROACH 6.1
6.2 SYSTEM CONFIGURATION 6.3
6.3 DEVELOP SYNTHETIC STRESS PATTERN 6.7
6.3.1 Introduction 6.7
6.3.2 Compliance Technical Analyses 6.7
6.3.3 Load Projections 6.8
6.3.4 Parameter Data for Stress Pattern 6.10
6.3.5 Flow Patterns 6.15
6.3.6 Projected 2031 Stress Pattern 6.17
6.3.7 Flow Projections 6.19
6.3.8 Load Projections 6.22
6.3.9 Performance Capacities of Unit Operations and Processes 6.24
6.3.10 Compliance Assessment 6.27
6.3.11 Summary and Discussion 6.31

7.0 CONCEPTUAL DEVELOPMENT 7.1
7.1 COLD WEATHER REQUIREMENTS 7.1
7.1.1 Cold Weather Issues 7.1
7.1.2 Recommendation 7.2
7.2 HEADWORKS 7.4
7.2.1 Raw Sewage Pumps 7.4
7.2.2 Screens 7.4
7.2.3 Grit Removal 7.5
7.2.4 Headwork Channel Modifications 7.7
7.3 CHEMICALLY ENHANCED PRIMARY TREATMENT 7.7
7.3.1 Primary Clarification 7.7
7.3.2 Flow Distribution and Calculation of Surface Overflow Rate (SOR) 7.8
7.3.3 Estimated Performance of CEPT 7.8
7.3.4 Chemical Feed System 7.9
7.3.5 Primary Sludge Removal System 7.10
7.4 INTERMEDIATE PUMPING STATION 7.10
7.4.1 Design Flows 7.11
7.4.2 Influent Flow Lines 7.11
7.4.2.1 Existing Primary Clarifier Flow 7.11
7.4.2.2 New High Rate Primary Clarifier 7.12
7.4.2.3 Backwash Clarifiers 7.12
7.4.3 Influent Pump Station Design Concept 7.13
7.4.4 Solids Handling 7.14
7.4.5 Influent Pump Station Sizing and Pumping Details 7.14
7.4.6 Lift Station Pump Control Strategy 7.15

Table of Contents

7.4.7 Lift Station Pumps 7.17
7.5 BIOLOGICALLY ACTIVE FILTRATION (BAF) 7.17
7.5.1 BAF Process....7.17
7.5.2 Key Design Criteria....7.19
7.5.3 Preliminary Process Sizing....7.20
7.5.4 Backwash Waste Management....7.21
7.5.4.1 Backwash Waste Flows and Characteristics....7.21
7.5.4.2 Existing Secondary Clarifiers....7.22
7.5.4.3 Proposed Modification of the Secondary Clarifiers for Backwash Clarification7.22
7.6 HIGH RATE CLARIFICATION (HRC) 7.24
7.6.1 HRC Process....7.24
7.6.2 Proposed High Rate Clarification System 7.26
7.7 EFFLUENT DISINFECTION....7.27
7.7.1 Impact of Bypassing Disinfection at Peak Flows 7.27
7.7.2 UV Disinfection System 7.29
7.7.3 Chlorination / Dechlorination System 7.31
7.8 BIOSOLIDS 7.32
7.8.1 Background 7.32
7.8.2 Biosolids Handling Processes 7.32
7.8.3 Sludge Blend Tank 7.33
7.8.4 Sludge Thickening....7.33
7.8.5 Anaerobic Digestion 7.33
7.8.6 Dewatering 7.34
7.8.7 Centrate Treatment 7.34
7.9 SITE DEVELOPMENT 7.34
7.9.1 Land Availability....7.35
7.9.2 Headworks....7.35
7.9.3 Side Stream Bypass Pipe....7.36
7.9.4 Primary Clarification 7.37
7.9.5 High Rate Clarification....7.37
7.9.6 Intermediate Pump Station 7.37
7.9.7 Biological Aerated Filter....7.38
7.9.8 Backwash Clarification 7.38
7.9.9 UV Disinfection....7.39
7.9.10 Chlorination / Dechlorination 7.39
7.9.11 Outfall 7.39
7.9.12 Biosolids 7.40
7.9.13 Future Effluent Requirements....7.41
7.9.14 Future Biological Phosphorous Removal 7.41
7.9.15 Geotechnical Investigations....7.42
7.10 HYDRAULIC PROFILE AND FLOOD PROTECTION 7.43
7.10.1 Hydraulic Model....7.43
7.10.2 Red River Levels 7.43
7.10.3 Hydraulic Profile 7.44
7.10.4 Main Stream Hydraulic Profile 7.44
7.10.5 Side Stream Hydraulic Profile....7.45

Table of Contents

7.10.6 50-Year Design....7.46
7.11 ADDITIONAL ITEMS REQUIRING FURTHER INVESTIGATION IN SUBSEQUENT DESIGN STAGES....7.47

8.0 CIVIL ASSET CONDITION ASSESSMENT....8.1
8.1 INTRODUCTION....8.1
8.2 PREVIOUS WORK AND SCOPE....8.1
8.3 2008 FACILITY ASSESSMENT....8.2
8.3.1 General Comments....8.3
8.3.2 Administration Building and Expansion....8.3
8.3.3 Pump and Screen Building....8.4
8.3.4 Grit Building....8.4
8.3.5 Maintenance/Service Building....8.5
8.3.6 Standby Generator Building....8.5
8.3.7 Odor Control Stack....8.6
8.3.8 Primary Clarifiers Nos. 1 and 2....8.6
8.3.8.1 Tanks Full Visual Inspection....8.6
8.3.8.2 Primary Clarifier No. 1 Tank Empty Visual Inspection....8.6
8.3.9 Primary Clarifier No. 3....8.7
8.3.9.1 Tank Full Visual Inspection....8.7
8.3.9.2 Primary Clarifier No. 3 Tank Empty Visual Inspection....8.7
8.3.10 Oxygen Reactors Nos. 1 and 2....8.7
8.3.11 Oxygen Reactors Nos. 3 and 4....8.8
8.3.12 PSA Building....8.8
8.3.13 Sludge Thickening and Sludge Truck Bay....8.8
8.3.14 Secondary Clarifiers Nos. 1 and 2....8.9
8.3.15 Secondary Clarifier No. 3....8.10
8.3.16 UV Building....8.10
8.3.17 Flushing Water System....8.10
8.3.18 Potable Water System Backflow Prevention....8.10
8.4 2011 FACILITY CONDITION ISSUES....8.11
8.4.1 General....8.11
8.5 INFORMATION GAPS....8.11
8.5.1 General....8.11
8.5.2 Building Issues....8.11
8.5.3 HPO Reactor Assessment....8.12
8.5.3.1 HPO Reactor Assessment Preparation....8.12
8.5.3.2 HPO Reactor Assessment Methodology....8.13

9.0 RISK ANALYSIS AND MITIGATION STRATEGIES....9.1
9.1 INTRODUCTION....9.1
9.2 RISK ASSESSMENT FRAMEWORK....9.1
9.3 PROJECT DRIVERS....9.2
9.4 RISK ASSESSMENT....9.3
9.5 SUMMARY....9.3

Table of Contents

10.0 REFERENCES 10.1

11.0 ACRONYMS AND ABBREVIATIONS 11.1

LIST OF TABLES

Table E.1 Effluent Compliance Criteria E.3

Table 2.1 Estimated Hauled Liquid Waste Volumes Received at the WE, SE, and NE Receiving Station Spanning 2005-201- 2.3

Table 2.2 Comparison of Year 2005 Volumes, Year 2008-2010 Volumes, and Year 2031 Design Basis Volumes 2.4

Table 2.3 Comparison of Master Plan Capital and O&M Cost Estimates for Options 1 and 6 (5) 2.5

Table 2.4 Updated 2011 Capital and O&M Cost Estimates for Options 1 and 6................2.7

Table 2.5 Summary of Hauled Wastewater Transportation Distance and Costs 2.8

Table 2.6 City and Hauler Cost Estimates for Options 1 and 6 ....2.9

Table 2.7 Qualitative Risk Assessment 2.10

Table 3.1 Effluent Criteria 3.2

Table 3.2 Worst Case Interpretation of Future Allowable Effluent Ammonia Loads and Limits – Chronic Monthly Load Limits (Applied as Daily Limit) 3.6

Table 4.1 Per Capita Wastewater Generation Rates (excluding Windsor Park) 4.7

Table 4.2 Current and Project Peak Flows to the SEWPCC Based on 6 Years of SEWPCC Flow Data (2005-2010, inclusive), Actual and Revised Per Capital Wastewater Generation, and Scaled Up Based on Population Growth Plus Additional Extraneous Flows from New Developments...................................4.15

Table 4.3 Summary of Historical Annual Average Population and Loads 4.17

Table 4.4 Summary of Calculated Historical Per Capita Loads (Annual Average Basis)..4.17

Table 4.5 Calculation of Per Capita Loading from HLW 4.19

Table 4.6 Projected AA Loadings (adjusted for HLW)..4.19

Table 4.7 Historical Seasonal Average Loadings – Raw Influent....................................4.21

Table 4.8 Historical Seasonal Maximum Month Loadings – Raw Influent .... *following page* 4.21

Table 4.9 Historical Seasonal Maximum Week Loadings – Raw Influent .... *following page* 4.21

Table of Contents

Table 4.10	Historical Seasonal Maximum Day Loadings – Raw Influent	4.23
Table 4.11	Load Factors – Ratio of Seasonal Average to Annual Average	4.24
Table 4.12	Load Factors – Ratio of Seasonal Max 30d to Annual Average	4.25
Table 4.13	Load Factors – Ratio of Seasonal Max 7d to Annual Average	4.26
Table 4.14	Load Factors – Ratio of Seasonal Max Day to Annual Average	4.27
Table 4.15	Summary of Seasonal Load Factors Relative to Annual Average Loadings	4.28
Table 4.16	Summary of Projected 2031 Projected 2031 Seasonal Loadings (based on a population of 270,000 peoples)	4.29
Table 4.17	Summary of Projected 2031 Seasonal Loadings (based on a population of 400,000 peoples)	4.30
Table 4.18	Summary of Current and Projected Wastewater Temperatures, Flows and Loads	*following page* 4.30
Table 4.19	Average Concentrations for Specific Parameters	4.31
Table 6.1	Effluent Compliance Requirements	6.1
Table 6.2	Summary of Influent Data Collected at the SEWPCC	6.10
Table 6.3	Ratio of Hourly Flow to Daily Average	6.16
Table 6.4	Comparison of SEWPCC Design Flows (ML/d) for a Projected Population of 270,000 People in the Year 2031	6.21
Table 6.5	Comparison of SEWPCC Design Loads (Kg/d) for a Projected Population of 270,000 People in the Year 2031	6.23
Table 6.6	Performance Values for Various Processes	6.24
Table 6.7	Maximum Allowable Effluent Ammonia Concentrations in August for Specific Flows	6.25
Table 6.8	NEWPCC Average Centrate Influent and Effluent Concentrations	6.27
Table 6.9	Maximum Predicted Values for Various Treatment Scenarios	6.29
Table 6.10	Estimated Year-Specific Ammonia Non-Compliances based on the Stress Pattern Analyses	6.29
Table 7.1	Screening Design Criteria	7.1
Table 7.2	Grit Removal Design Criteria	7.6
Table 7.3	Estimated Removal Efficiencies for CEPT	7.9
Table 7.4	Proposed Pump Speed and Level Control Strategy for Design Year 2031	7.16
Table 7.5	Proposed Pump Speed and Level Control Strategy for Design Year 2061	7.17
Table 7.6	Summary of Dirty Backwash Water Clarification Design	7.23
Table 7.7	HRC Influent Design Criteria	7.26
Table 7.8	HRC Removal Efficiencies	7.26

Table of Contents

Table 7.9	Existing UV Disinfection System Design Criteria	7.29
Table 7.10	Red River Elevations at the SEWPCC Outfall and Return Period	7.30
Table 7.11	Proposed UV Disinfection System Design Criteria	7.30
Table 7.12	Proposed Chlorine Disinfection System Design Criteria	7.31
Table 7.13	Red River Elevations at the SEWPCC Outfall for Various Return Periods	7.43
Table 7.14	Proposed Capacity for Main Stream Processes	7.44
Table 7.15	Proposed “Out of Water” Elevation for Design River Level (230.00)	7.45
Table 7.16	Proposed Capacity for Side Stream Processes	7.46
Table 7.17	Additional Items Requiring Investigation at Subsequent Design Stage	7.47
Table 9.1	Assessing Likelihood/Probability of Risk Occurrence	*following* 9.3
Table 9.2	Assessment Framework to Quantify Risk and Opportunity	*following* 9.3
Table 9.3	Framework to Assessment Risk Severity	*following* 9.3
Table 9.4	Risk Register	*following* 9.3

LIST OF FIGURES

Figure 1.1	Layout of Existing Facility	1.1
Figure 4.1	Generalized Approach to Estimating Flows and Loads to SEWPCC	4.4
Figure 4.2	Projected SEWPCC Population	4.5
Figure 4.3	Historic Winter Dry Weather Flows to SEWPCC	4.7
Figure 4.4	Residential Per Capita Demand	4.8
Figure 4.5	Approach Used to Estimate Projected 2031 Dry and Wet Weather Flow to SEWPCC	4.9
Figure 4.6	Schematic showing SEWPCC Collection System	4.11
Figure 4.7	Event-specific Estimate of Emergency Sanitary Sewer Overflows (SSO) and Wet Weather Flows Received at the SEWPCC	4.13
Figure 4.8	Event-specific Estimate of Emergency Sanitary Sewer Overflows (SSO) and Wet Weather Received at the SEWPCC based on Projected 2031 Population of 270,000 People	4.14
Figure 5.1	Mass Balance Diagram Maximum Flow – Spring	*following* 5.2
Figure 6.1	Schematic of Upgraded and Expanded SEWPCC Processes and Unit Operations	6.5
Figure 6.2	Preliminary Flow Schematic for 2031	*following* 6.8
Figure 6.3	Trend and Correlation Analysis for TSS	6.11

Table of Contents

Figure 6.4 Trend and Correlation Analysis for Nitrogen 6.12
Figure 6.5 Trend and Correlation Analysis for Effluent Ammonia.... 6.13
Figure 6.6 Trend and Correlation Analysis for Phosphorus.... 6.14
Figure 6.7 Trend and Correlation Analysis for BOD/TOC/COD.... 6.15
Figure 6.8 Month-Specific Average-Day Diurnal Flow Pattern at the SEWPCC 6.17
Figure 6.9 Comparison of Projected Flow Patterns to SEWPCC 6.19
Figure 6.10 Estimated Ammonia Non-Compliances based on the Stress Pattern Analyses for Scenario 1 .... 6.30
Figure 6.11 Estimated Ammonia Non-Compliances based on the Stress Pattern Analyses for Scenario 2 .... 6.30
Figure 7.1 Partially Covered Building over BAF Tanks 7.3
Figure 7.2 Covered Building in BAF Plant and Boisbriand, Quebec.... 7.3
Figure 7.3 BAF Plant in Norway 7.4
Figure 7.4 Headworks Channel Concepts .... *following* 7.6
Figure 7.5 Proposed CEPT Schematic.... *7.7*
Figure 7.6 Site Plan – 2031 Flows.... *following* 7.12
Figure 7.7 Intermediate Pumping Station .... *following* 7.12
Figure 7.8 2031 Predicted Geometric Mean E.coli Concentrations in Final Effluent after Disinfection of Flows up to 325 ML/d.... 7.28
Figure 7.9 Site Plan .... *following* 7.34
Figure 7.10 Land Availability .... *following* 7.36
Figure 7.11 Main Stream Hydraulic Profile – 420 MLD for 2031 Design .... *following* 7.36
Figure 7.12 Side Stream Hydraulic Profile – 420 MLD for 2031 Design.... *following* 7.36
Figure 7.13 Preliminary Flow Schematic .... *following* 7.44
Figure 8.1 Site Plan .... *following* 8.1

Table of Contents

APPENDICES

APPENDIX A	Hydraulic Modeling Output
APPENDIX B	Condition Assessment Photographs
APPENDIX C	Project Risk and Opportunity Register
APPENDIX D	Data from SEWPCC PDR, Wet Weather Treatment
APPENDIX E	Biosolids Calculations
APPENDIX F	Topographic Site Survey

1.0 Introduction

1.1 INTRODUCTION

The City commissioned the South End Water Pollution Control Centre (SEWPCC) in 1974 to treat wastewater collected from the south end sewerage catchment of Winnipeg. The SEWPCC was expanded in 1993 to increase capacity and improve process reliability. UV disinfection and computerized work management systems were added in 1999 and 2003 respectively.

The SEWPCC unit processes consist of raw sewage pumping, screening, grit removal, primary clarification, high purity oxygen (HPO) generation with pressure swing adsorption units (PSA), high purity oxygen activated sludge, secondary clarification, UV disinfection, sludge storage and hauling to the NEWPCC. Other support functions include odour control, stand-by power generation and a septage receiving station. A layout of the existing facility is shown on Figure 1.1.

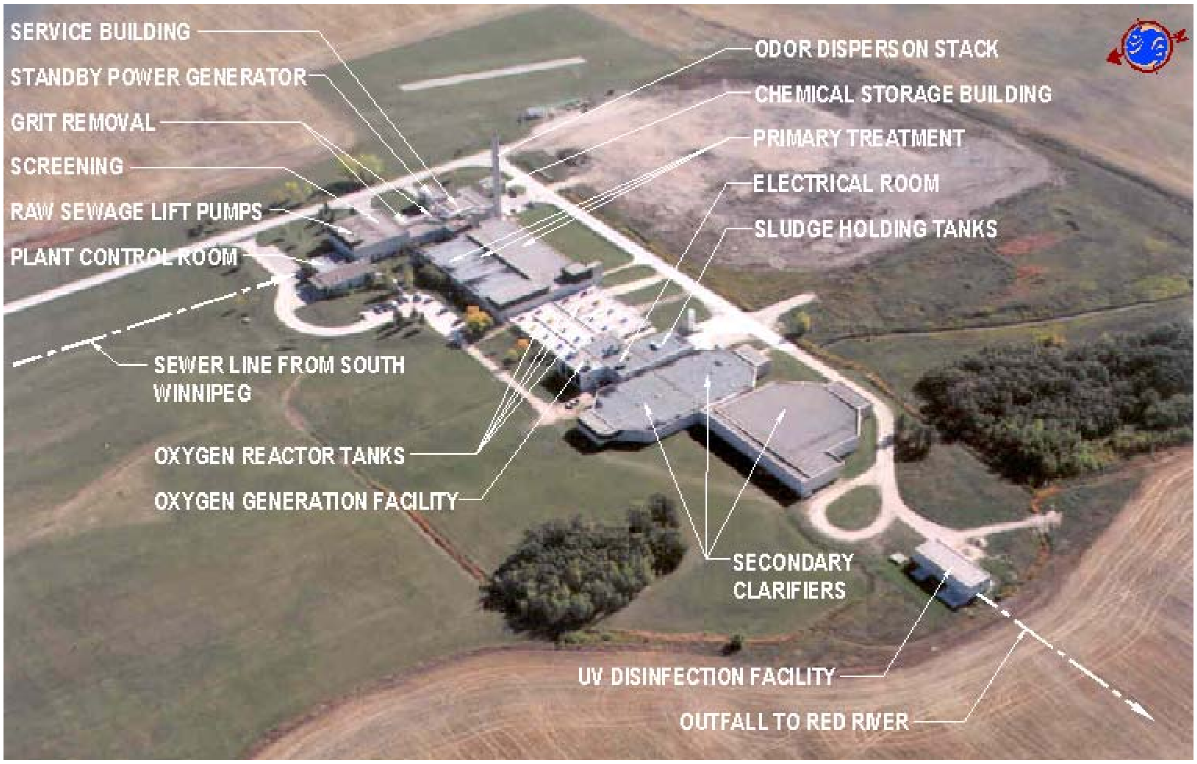


Figure 1.1: Layout of Existing Facility

Wastewater loading to the SEWPCC has reached its existing design capacity. Continued development pressure in the SEWPCC service area, including relatively new developments such as Waverley West, Island Lakes, Sage Creek and Dawson Trail, necessitates that the

SEWPCC be expanded to accommodate the increased wastewater loading to the plant. At the same time, there is growing concern regarding excessive nutrient loadings to Lake Winnipeg from within its watershed, including nutrients in treated effluents from Winnipeg, is contributing to the eutrophication of the lake. To address environmental and public health concerns, Manitoba Conservation is imposing more stringent effluent criteria on Winnipeg's three wastewater treatment plants, including the SEWPCC. Manitoba Conservation issued Environmental Licence No. 2716R for the SEWPCC on June 19, 2009.

Stantec was retained by the City in 2006 to investigate means to expand and upgrade the SEWPCC. Stantec delivered the SEWPCC Upgrading / Expansion Preliminary Design Report (Stantec, 2008) and SEWPCC Upgrading Expansion Conceptual Design Report (Stantec, 2009). Subsequent to these studies, the City entered into a partnership with Veolia Water to upgrade and expand the City's SEWPCC and North End Water Pollution Control Centre (NEWPCC). The combined City and Veolia team is referred to as the Program Team in this report.

Veolia subsequently produced a document entitled Winnipeg Sewage Treatment Program South End Plant - Process Selection Report (PSR, 2011) to select the process to be used to upgrade and expand the SEWPCC. The Program Team subsequently retained Stantec to confirm the design criteria, validate that the process proposed by the Program Team has the capability to meet the design criteria, and identify key information gaps that must be addressed in the Preliminary Design phase. Specifically, the information gained from this assignment will help to confirm a Biologically Active Filtration (BAF) system in concert with a chemical-based phosphorus (Chem-P) removal system can comply with effluent criteria set by the Program Team; and to define the scope of work for later stages of design.

In addition, Stantec was asked to update the Hauled Liquid Waste (HLW) business case with newly available, current information to determine if the construction of a new HLW facility at the SEWPCC remains the preferred overall solution for the City.

1.2 PROJECT OBJECTIVES

The objectives of the project are summarized below.

- **Selected Process**
  - The design is to consist of selected a biologically active filtration (BAF) process with nitrification and denitrification capabilities in combination with a chemical phosphorus removal process, and a side-stream high-rate clarification (HRC) process for the upgrade and expansion of the SEWPCC.
  - In addition, provide sufficient space and process configuration for possible conversion to a biological phosphorus removal process.

- **Biosolids Implementation Strategy**
  - Perform a mass balance on the maximum month flows and loads for 2031 to estimate solids production and approximate the solids treatment requirements (i.e., digestion and dewatering).
  - Estimate the recycle flows and loads from filtrate and centrate.
- **Hauled Liquid Waste (HLW) Strategy**
  - Update the business case based on more current information to determine if the addition of a HLW acceptance facility at the SEWPCC remains the preferred option.
- **Effluent Requirements**
  - Assess the ability of the plant upgrades to achieve a specified effluent quality based on criteria provided by the Program Team.
- **Validation of Flows and Loads for Design**
  - Update the flows and loads based on more recent and complete wastewater data for projected populations associated with the design horizons of 2031 and 2061.
  - Incorporate flood protection requirements in the hydraulic design of the upgraded plant.
  - Identify emerging regulatory trends and possible design provisions that might be necessary in the future to accommodate more stringent effluent discharge limits.
- **Plant Concept Development**
  - Develop functional aspects to be considered and expanded upon in subsequent design phases.
  - Define the major process components and identify feasible layouts on the SEWPCC site.
  - Develop site plans to approximate scale depicting the integration of unit operations and processes for projected flows and loads based on a BAF treatment process with HRC and a chemical based phosphorus removal system, and develop a second plan for a biological based phosphorus removal system.
- **Land Surveys and Geotechnical Investigations**
  - Undertake topographical, property surveys and supplemental geotechnical investigations as required to support the site build out of the upgraded facility. (Appendix F – Site Survey)

- **Asset Condition Assessment**
  - Update the civil asset assessment done by Stantec in 2008 to identify information gaps that should be addressed as part of this assignment and those that can be dealt with in subsequent engineering phases.
  - No assessment to be conducted on the condition of the mechanical and electrical assets since this work is being coordinated by the Program Team and completed by others.
- **Major Equipment Procurement Packages**
  - Prepare equipment supply contract documents for the procurement of BAF and HRC treatment equipment. (Due to the size and complexity of the procurement document, it has been provided for review under separate cover).
- **Risk Analysis and Mitigation**
  - Conduct a risk assessment on aspects that could negatively impact the successful delivery of the project and formulated a prioritized mitigation strategy based on the severity of the risk and timing of the risk.

1.3 REPORT DEVELOPMENT

Stantec coordinated a series of review meetings to present information and findings associated with each task to the Program Team as work progressed. The meetings were intended to highlight the analyses and findings for review and discussion by the Program Team in order to confirm information to be contained in technical memorandums. The direction and comments received from the Program Team were used to clarify the information presented, undertake additional review/analyses as required, or conclude the deliverable. Individual draft technical memoranda were prepared and provided to the Program Team as key tasks were completed for collective review and comment. As engineering progressed, new information became available and was used in subsequent technical evaluations and incorporated into later technical memorandums. It is important to note that the earlier individual reviews and assessments associated with each task were sufficiently accurate to form a sound technical basis for subsequent interactions of analyses and design validation.

This report consolidates the draft technical memoranda into a comprehensive document and partially restructures the content to permit the flow of information in a more logical format for project definition and validation efforts. To achieve clarity, this report contains standardized and updated values, as well as recalculated technical analyses where appropriate for reasons of precision. The information contained in this report supersedes that contained in the draft technical memoranda.

2.0 Hauled Liquid Waste Strategy

2.1 PURPOSE

Stantec has been retained to update the business case for hauled liquid waste (HLW) received at the SEWPCC and NEWPCC. Additional technical and financial information has become available for analysis since the construction of a new HLW acceptance facility at the NEWPCC. The Program Team requested that the HLW business case and associated factors used in the assessment process be updated at this time because a decision is required regarding the acceptance and treatment of HLW at the SEWPCC due to its possible technical design implications on the treatment processes and overall construction schedule and costs.

Factors leading to the review of the 2007 HLW business case include:

- A 2008 wastewater characterization study at the SEWPCC found that HLW negatively impacted the nitrification growth kinetics;
- Better and more accurate financial information associated with the construction of a HLW facility at the NEWPCC; and,
- Revised data is now available regarding the amount of HLW received at both the NEWPCC and SEWPCC since 2007 and the influence this data has on projected loads for the future.

This decision impacts the haulers that use the SEWPCC receiving station facility (estimated at one-third of the haulers), as well as the treatment process design. Undertaking the business case requires the completion of technical, financial, and risk analyses to compare the options of continuing to receive hauled liquid waste at the SEWPCC verse diverting all HLW to the NEWPCC (referred to as Option 6 in the 2007 study). The results of this analysis will confirm the appropriate approach to HLW acceptance at the SEWPCC based on an updated business case.

2.2 BACKGROUND

Stantec completed a study in 2007 titled “Winnipeg Regional Hauled Wastewater Plan” for the City of Winnipeg. The objective of the study was to identify and rank alternatives for receiving and treating the hauled liquid waste generated within the City of Winnipeg and surrounding municipalities. Of the many alternatives investigated, two were eventually identified as preferred options:

- Option 1 – Close the West End (WE) hauled waste receiving station and add minimum upgrades at the North End (NE) and South End (SE) hauled waste receiving stations
- Option 6 - Close the WE and SE hauled waste receiving stations and make appropriate upgrades at the NE station

The basis for the evaluation of these options was historic operating data available (2005-2007) and did not include the impacts of closing the WE hauled waste receiving station. A key assumption made at the time of the analysis was that the WE hauled waste flows would be directed in approximately equal amounts to the SE and NE receiving stations. Economic analyses indicated that both options had similar financial implications. Furthermore, waste haulers were consulted as stakeholders through surveys, newsletters and a public meeting in order to gather additional feedback and determine general acceptance of the proposed plans. Based on the economic and technical evaluations, Option 1 was recommended. The haulers indicated that having two receiving locations provided them with more operational flexibility, lower operating costs, and reduced transportation distances (haul time).

In 2008, the City undertook a wastewater characterization study at the SEWPCC. The study determined that HLW negatively impacted the nitrification growth kinetics, which would necessitate either pre-treatment of the HLW or an increase in size of the secondary bioreactor tanks to achieve the required level of treatment. This requirement would significantly increase the cost of receiving and treating HLW at the SEWPCC. As the HLW plan is based on a user-pay framework, this new requirement has the potential to alter the HLW business case which formed the basis of the 2007 Plan.

The City is currently moving forward with the Conceptual Design for SEWPCC upgrades and expansion. It is therefore critical that a long-term HLW management plan for the City be finalized at this time. The City decided to update the HLW business case in order to address the new information related to the impacts of HLW on the treatment process, and to validate whether or not the WE flow split assumptions made in 2007 held true.

The remainder of this report is focused on:

- Confirming Master Plan hauled waste flow projections/assumptions using more recent hauled waste information
- Updating capital/O&M/recovery costs of implementing Option 1 (SE and NE hauled waste receiving) versus Option 6 (NE hauled waste receiving) given newly available information – i.e., completing the City's "business case"
- Updating data hauler impacts such as travel distance and related costs
- Developing a preliminary risk management strategy

2.3 TECHNICAL AND FINANCIAL ANALYSES

Technical and financial analyses are provided in this section in order to compare Option 1 and 6 using newly available information that did not exist at the time the Master Plan were completed in 2007.

2.3.1 Hauled Waste Flows

Available hauled liquid waste volume information spanning 2005-2010 was reviewed in order to confirm master plan flow projections/assumptions made in 2007 (see Table 2.1).

Table 2.1 – Estimated Hauled Liquid Waste Volumes Received at the WE, SE, and NE Receiving Station Spanning 2005-2010

Year	"Gate" Hauled Liquid Waste Volumes Received at City Plants (1)					"Gate" Volumes Reduced 20%	
	SE (m^3/yr)	NE (m^3/yr)	WE (m^3/yr)	Total (m^3/yr)	Total (m^3/d)	Total (m^3/yr)	Total (m^3/d)
2005	29,596	67,337	39,258	136,191	373	108,952	299
2006 (2)	5,219	80,625					
2007 (3)	42,012	85,016					
2008	59,105	96,434		155,539	426	124,430	340
2009	51,161	99,667		150,828	413	120,662	330
2010 (4)	40,750	99,570		140,320	384	112,256	308
2008-2010 average	**50,000**	**98,000**		**148,000**	**405**	**118,400(5)**	**325**

Notes:
1. "Gate" volume is the sum of the tanker truck volumes received at the plant for the year. Actual volumes have been reduced by 20% to account for tankers being partially filled to approximately 80% on average.
2. Incomplete dataset for year 2006 hence no totals estimated.
3. Hauled waste receiving ceased at WE plant in July 2007, hence dataset is uncertain.
4. Year 2010 dataset was available for 11 months (Jan-Nov). Stated volumes were increased by factor 12/11 to estimate full year 2010 volumes.
5. Total gate volume based on 40,000 (m^3/yr) from SE and 78,400 (m^3/yr) from NE.

It is evident from the data shown in Table 2.1 that the volumes of HLW to the SE plant have declined in the recent years. The City indicated that a possible reason could be the increase in fee charged to the haulers for the non-household waste, resulting in the haulers taking their loads to other municipal facilities outside of Winnipeg. The City added that a current rate study has recommended a further significant increase in hauling fees for non-household waste, which may result in a further reduction in non-household waste being received at the NE and SE. However, this trend could quickly reverse if the other municipalities raise their rates as well. There is a strong possibility that future HLW volumes may decline in response to the anticipated fee increases associated with non-household waste. However, for conservatism in this business case, the projections were kept in line with the 2007 assumptions. A detailed discussion on the projections was provided in Section 3.7 of the 2007 report. This data is summarized in Table 2.2.

The 2007 study reviewed hauled liquid waste volumes received in year 2005 at the SE, WE, and NE receiving stations and then made projections to estimate the hauled waste volumes to be expected over the next 25 year period.

Table 2.2 – Comparison of Year 2005 Volumes, Year 2008-2010 Volumes, and Year 2031 Design Basis Volumes

Receiving Station	Year 2005 Volume (m^3/yr)	Year 2008-2010 Volume (m^3/yr)	Year 2031 Design Basis Volume (m^3/yr)
SE	23677 (22%)	40000 (34%)(1)	30660+ (50% WE of 35405) =48363 (36%)
NE	53870 (49%)	78400 (66%)(1)	67890+ (50% WE of 35405) =85593 (64%)
WE	31406 (29%)	-	-
Total	108,953 (~300 m^3/d)	118,400 (~325 m^3/d)	133,956 (~367 m^3/d)

Notes:
1. Values are gate volumes calculated at 80% of the actual volumes to account for partially filled tankers.

Key findings to note with relevant to recent 2008-2010 operation verse the 2007 study projections are:

- Hauled waste volumes have increased marginally from 2005 through years 2008-2010. The increase appears to be less than 10% (from ~300 m^3/d to ~325 m^3/d). However, there is a strong potential for the volumes to be lower in the future, as discussed earlier.
- The original assumption that WE flows would split 50/50 between SE and NE plants once it closed appears to be accurate. Actual 2008-2010 data finds a 34/66 flow split between SE/NE plants, which compares favorably with a 36/64 flow split as assumed in the 2007 study for the 2031 design year.
- Using a design flow of 367 m^3/d as a basis for evaluating the hauled liquid waste alternatives in the master plan would appear to be reasonable given the marginal increase in flows from 2005 to 2010.

2.3.2 Financial Impacts

2.3.2.1 City Costs

Financial estimates to implement Options 1 (SE and NE receiving stations) and 6 (NE receiving station only) were made in the 2007 study based on the available information in 2006/2007. The master plan capital and O&M cost estimates for Options 1 and 6 are summarized in Table 2.3 and were based on receiving 367 m^3/d hauled liquid waste.

Table 2.3 – Comparison of Master Plan Capital and O&M Cost Estimates for Options 1 and 6 (5)

Option	Capital Cost (1)	Annual O&M Cost (2)	25yr NPV O&M (3)	Cost Recovery = Basic fee + surcharge ($/6.8m^3 load) (4)
1 (SE +NE)	$2.7M	$302k	$4.14M	$32.44
6 (NE only)	$2.5M	$278k	$3.82M	$31.22

Notes:

1. Opinion of probable cost with ±35% accuracy; based on defined scopes of work developed in master plan; year 2009 construction costs; includes 25% contingency and 15% engineering.
2. Opinion of probable cost with ±35% accuracy; based on defined scopes of work developed in master plan. Note that the annual O&M costs also include a cost recovery portion for capital replacement.
3. 25 year net present value of O&M with 5.25% interest rate and 1.9% inflation rate.
4. The estimated charge required to recover costs for the receiving stations upgrades; based on 367 m^3/d hauled waste, $17.07 basic receiving charge for average tanker load of 6.8 m^3; $2.3/m^3 surcharge for option 1 and $2.1/m^3 surcharge for option 6.
5. Costs associated with scopes of work as defined in the Master Plan.

Since completing the Master Plan analysis in 2007, additional information has been generated along with new HLW infrastructure being implemented by the City. Three changes impact the financials associated with implementing Option 1 and 6. The impacts include:

- **Completing a detailed design for the SE hauled waste receiving station** – A new hauled waste receiving station was designed for the SE plant with additional features and more automation than was specified in the 2007 study. The pre-tender estimate for construction in 2010 was $1.9M.
- **Process modeling at SEWPCC to determine hauled waste impact on treatment process** – Through a wastewater characterization study completed in 2008, it was determined that hauled liquid waste would impair the ability of the new plant to nitrify and that the bioreactor would need to be increased in size to accommodate the receiving of hauled liquid waste. The cost impact to the treatment process to accommodate the acceptance of hauled liquid waste was originally estimated at $4.65M in 2007 and adjusted to $5.1M in 2010 dollars. It should be noted that given the unknown nature of the substances in the HLW that negatively impacted nitrification during the characterization study it was determined that increasing the bioreactor size provided a safer and more reliable solution than implementing any pre-treatment at the SE facility. This determination was based on a suspended growth system utilizing an Integrated Fixed-film Activated Sludge (IFAS). Specifically, without knowing what substance or substances were causing the inhibition, it could not be concluded that a conventional physical-chemical pretreatment system would effectively remove the substance causing the impaired nitrifier growth. With

this understanding, the nitrification zone of the suspended growth activated sludge biological nutrient removal (BNR) system was increased in size to compensate for the possible impairment of nitrifier growth to unknown substances in the HLW. It is believed that a similar impairment would be observed in a BAF based treatment system and that appropriate design approach would be required to compensate for the reduced nitrification activity associated with a fixed film process. This would translate into a longer hydraulic retention time and associated tankage required for this purpose. As nitrifier growth impairment is a dose-response relationship, reducing the dose by means of dilution can be an effective method to reduce impairment below the dose threshold that causes a measureable response. Since the North End Water Pollution Control Center (NEWPCC) is about three times the flow observed at the SEWPCC, the NEWPCC can provide significantly more dilution and minimize the need for balancing storage to reduce the impairment. As such, additional pre-treatment costs were not included at the SEWPCC.

- **Upgrading the NE hauled waste receiving station** – One additional receiving lane was constructed in 2010/2011 in an upgrade for receiving hauled liquid waste at the NE (making two lanes total for hauled waste at the NE). The cost for this upgrade to accommodate hauled waste was $1.7M. It is estimated that addition of another lane at the NEWPCC for hauled waste (to 3 lanes available per the 2007 study), would require an additional $1.7M.
- NE plant impacts related to hauled waste – Based on the SE waste characterization information there is a possibility that NE plant performance could be impacted by increased volume of hauled waste. Since the secondary process upgrades are unknown at his time, it was assumed that implementing pre-treatment will mitigate any impact of receiving higher volumes of HLW. Since the NEWPCC receives about three times the wastewater flow received at the SEWPCC, potential inhibition on nitrifiers from HLW would be less at the NEWPCC. Reduction in potential inhibition of nitrifiers is due to the greater protection afforded by dilution of toxic substances in the HLW that could negatively impact ammonia oxidizing bacteria growth. As such, it is expected that less mitigative measures would be required at the NEWPCC to protect the biological processes. A more robust treatment condition at the NEWPCC favors Option 6. Hence, costs associated with pre-treatment were included based on the 2007 study estimate, to provide flow equalization and primary clarification. The 2007 Option 4 cost of approximately $1.9M was increased to $3.1M for Option 1 and $4.0M for Option 6 after adjusting for flows and inflation.

Table 2.3 values have been revised to account for recent City activities/findings related to hauled liquid waste receiving at the SE and NE plants with updated values presented in Table 2.4.

Key findings to note with respect to financial implications to the City associated with implementing Option 1 (SE + NE receiving) verse Option 6 (NE receiving) include:

- The revised capital and O&M cost estimates for Options 1 and 6 have changed significantly from the 2007 Master Plan. The key difference is: 1) the need to increase the bioreactor sizes at the SE plant to accommodate hauled liquid waste estimated at $5.1M for Option 1 and 2) the potential need to add flow equalization and primary clarification at the NE plant to receive all hauled waste estimated at $4.0M (for Option 6) and $3.1M (for Option 1).

- The charges levied to commercial haulers to allow the City to recover O&M costs will be significantly higher for Option 1 verse Option 6 – now estimated at $86 verse $60 per 6.8 m^3 load for Options 1 and 6 in 2007.

2.3.2.2 Hauler Costs

The 2007 study included estimations for hauled waste water transportation distance and costs for the six short-listed alternatives including Options 1 and 6. These are summarized below in Table 2.5.

It should be noted that Transport Canada has not updated the unit cost per Truck Kilometer of Travel (TKT). Also considering that diesel prices have remained relatively stable since 2006 and considering minor increase in salaries but better fuel efficiencies in the trucks, the average annual transportation cost was kept at the 2005 level of $1.78/TKT.

Table 2.4 – Updated 2011 Capital and O&M Cost Estimates for Options 1 and 6

Option	Capital Cost (1)	Annual O&M Cost (2)	25yr NPV O&M (3)	Cost Recovery = Basic fee + surcharge ($/6.8$m^3$ load) (4)
Current				$17.07 (for household waste)
1 (SE +NE)	$1.9M (SE new waste station) $1.7M (NE new 2010/2011 upgrades) $5.1M (SE bioreactor vol.) $3.1M (NE flow EQ+PC) Total= $11.8M	$1,361k ($859k)	$18.7M	$17.07+69.09 = $86.16
6 (NE only)	$1.7M (NE new) 2010/2011 upgrades) $1.7M (NE Proposed 3rd receiving lane) $4.0M (NE flow EQ + PC) Total = $7.4M	$847k ($539k)	$11.6M	$17.07+43.0 = $60.07

Notes:
1. Capital cost estimate assumptions:
   a. Option 1 - SE new waste station 2010 pre-tender estimate of $1.9M; $1.7M for current HLW upgrades at the NE; 2010 cost estimate of $5.1M to increase SE bioreactor volume to accommodate hauled liquid waste receiving and $3.1N to add EQ+PC at NE.
   b. Option 6 – Estimated $1.7M cost to HLW upgrades at NE, $1.7M to add a third receiving lane at NE; estimated $4.0M cost to add flow equalization and primary clarifier at NE based on a $1.9M master plan cost estimate to add flow equalization and primary clarification to SE plant (option 4).
   c. Opinion of probable cost with ±35% accuracy; based on defined scopes of work; year 2011 construction costs; includes 25% contingency and 15% engineering.
2. O&M cost estimate assumptions:
   a. Un-bracketed term represents the equal annuity to be collected over 25 years to recover the costs for operation (labor, heating, electricity) and infrastructure replacement.
   b. Bracketed term represents the capital recovery for infrastructure replacement.
   c. Opinion of probable cost with ±35% accuracy; based on defined scopes of work.
3. 25 year net present value of O&M with 5.25% interest rate and 1.9% inflation.
4. The estimated charge required to recover costs for the receiving stations upgrades; based on 367 m^3/d hauled waste, $17.07 basic receiving charge for average tanker load of 6.8 m^3; $10.2 surcharge for option 1 and $6.3 surcharge for option 6.

Table 2.5 – Summary of Hauled Wastewater Transportation Distance and Costs

Option	2007 Master Plan Findings			2011 Estimates
	Average # of trips per day	Average Annual TKT (1)	Average Annual Transportation Cost (2)	Average Annual Transportation Cost (3) (rounded)
Base (SE+WE+NE)	39	118,070	$210,164	-
1 (SE +NE)	39	110,771	$197,172	~$200,000
6 (NE)	39	119,681	$213,032	~$215,000

Notes:
1. Average annual Truck Kilometers of Travel (TKT) was estimated at 118,070 km in 2005 with receiving at SE, WE, and NE plants.
2. Average annual transportation cost estimated as TKT x $1.78/TKT in 2007. The $1.78 rate was taken from Transport Canada 2005 data.
3. Average annual transportation cost in 2011 was assumed similar to 2007 given similar diesel price, better truck fuel efficiency, and minimal labor increase.

Key findings to note with respect to hauler transportation distance and costs associated with implementing Option 1 (SE + NE receiving) verse Option 6 (NE receiving) include:

- Total Truck Kilometers of Travel (TKT) differs by less than 10 percent between Options 1 and 6 according to the analysis completed in the master plan.
- Assuming similar unit operating cost per TKT, then truck hauling operating costs will differ by less than 10 percent between Options 1 and 6.

In 2006, the Water and Waste Department licensed 73 wastewater hauling vehicles. With an average volume of hauling truck capacity of 10.6 KL, the annul number of truck kilometers of travel (TKT) was estimated at 118,070 km. In comparison, the TKT for the City of Winnipeg was estimated at 100 million km. Based on this information, the 2007 HLW transport with Winnipeg constitutes about 0.1% of the total annual TKT.

2.3.2.3 Total Costs (City + Hauler)

An estimate of the total costs charged to hauled waste generators for City treatment and hauler transportation/profit have been summarized in Table 2.6 for Options 1 and 6.

Table 2.6 – City and Hauler Cost Estimates for Options 1 and 6

Option	Capital Cost (1)	Annual O&M Cost (2)	25 yr NPV O&M (3)	Total Charges = City (Basic fee + surcharge) + trucking charge ($/6.8m^3 load) (4)
1 (SE +NE)	$1.9M (SE new waste station) $1.7M (NE 2010/2011 upgrades) $5.1M (SE bioreactor vol.) $3.1M (NE flow EQ+PC) Total= $11.8M	$1,621k ($859k)	$22.3M	$17.07+69.09+13.20 = $99.36
6 (NE only)	$1.7M (NE 2010/2011 upgrade) $1.7M (NE 3rd receiving lane) $4.0M (NE flow EQ + PC) Total = $7.4M	$1,127k ($539k)	$15.5M	$17.07+43.0+14.21 = $74.28

Notes:
1. Capital cost estimate assumptions (refer to Table 2.4).
2. O&M cost estimate assumptions (refer to Table 2.4).
3. 25 year net present value of O&M with 5.25 percent capital cost and 1.9 percent inflation.
4. The estimated total charge to hauled waste generators based on: 367 m^3/d hauled waste, City charges for basic fee and surcharge to recover their capital/O&M costs, and commercial hauler charges for trucking and profit.

Trucking charges will be similar for Option 1 and 6. Trucking charges will add approximately $14 per 6.8 m^3 tanker load based on a unit trucking cost of $1.78 per TKT and distance estimates made in the 2007 study.

2.4 RISK ASSESSMENT

The 2007 hauled liquid waste Master Plan did not consider relative risks associated with each of six short-listed options. A risk matrix is presented in Table 2.7 for options 1 and 6 in order to compare and highlight some of the relative risks (see Table 2.7).

Table 2.7 – Qualitative Risk Assessment

Risk Factor/Impact of Risks	Option 1 – (SE + NE)	Option 6 (NE only)
Main Process Upset – i.e., cause extended period of process upset	Higher Risk (Negative) • Mitigation includes: limit HLW flow to the SE plant, including automatic sampling; implement flow equalization and primary clarification at the NE.	Lower Risk (Positive) • Mitigation includes: limit HLW loading, flow equalization & primary clarification
Effluent Permit Violations – i.e., cause short duration effluent exceedances	Higher Risk (Negative) • Mitigation includes: limit HLW to the facilities, implement automatic sampling for both SE/NE and construct flow equalization and primary clarification at the NE.	Lower Risk (Positive) • Mitigation includes: limit HLW relative loading, flow equalization & primary clarification
Major Traffic interruption near the NE – i.e., flexibility to manage interruptions to receiving	Lower Risk (Positive) • Mitigation includes: two receiving stations allows some diversion opportunity	Lower Risk (Negative) • Mitigation includes: developing alternative traffic management routes to get HLW back to the NEWPCC facility.
HLW Spill During Transportation	Medium Risk (Negative) • Mitigation includes: City enforcing that the haulers take a defensive driving course and have an Emergency Spill Response Plan.	Medium Risk (Negative) • Mitigation includes: City enforcing that the haulers take a defensive driving course and have an Emergency Spill Response Plan.

Investigations to date, suggest the risk of process upset and effluent permit violations are greatest at the SEWPCC because the relative loading of the hauled liquid waste at the SEWPCC is higher than if all the hauled liquid waste were to be received at the NEWPCC. Maintaining process stability and consistent effluent quality is the key risk factor.

Lesser risk factors include aspect such as traffic interruptions. Option 1 (SE+NE) having two receiving stations will provide greater flexibility to the haulers than Option 6 (NE). Short duration service interruptions could be managed by limiting receiving to maximum levels at the “OPEN” station. Based on directions received from the Program Team, maintaining the existing HLW receiving station at the SEWPCC for Option 6 for only emergency situations would be problematic and consequently not practical for implementation as an option.

2.5 CONCLUSIONS AND RECOMMENDATIONS

Key conclusions arising from this analysis includes:

- The capital and O&M costs associated with implementing Option 6 are lower than Option 1. Capital cost estimates are estimated at $7.4M and $11.8M, respectively, and 25 year net present value (NPV) of O&M costs are estimated at $11.6M and $18.7M, respectively.
- The City will need to charge haulers more to recover the higher capital and O&M costs associated with Option 6 and 1, estimated at $60 and $86 respectively per typical 6.8 m^3 load.
- Hauler transportation distance and costs are marginally higher for Option 6 than 1. Transportation distance and costs are estimated to be 8 percent higher for Option 6 over 1.
- Trucking costs will be similar for Options 6 and 1 – respectively estimated at $14 and $13 per 6.8 m^3 load. This will increase the total costs borne by hauled waste generators to $74 and $99 per typical 6.8 m^3 load for Option 6 and 1 respectively.
- The risk associated with treating hauled waste within the main treatment process is greater at the SEWPCC than at the NEWPCC. This favors Option 6.

The key recommendations arising from this analysis includes:

- Develop a plan to implement Option 6:
  - From an economic standpoint, the cost to the City and hauled waste generators is less to receive hauled waste at the NEWPCC exclusively. This will result in lower hauler charges and lower costs for users dependent on hauled waste removal services.
- Review findings with affected stakeholders:
  - This includes consulting with haulers and generators to inform them of recent findings and proposed plan.
- For the purpose of this study it was assumed that any impact on the future BNR upgrade at the NE can be mitigated through flow equalization and primary clarification of the HLW.

3.0 Regulatory Framework

3.1 BACKGROUND

The upgrade of the SEWPCC is driven by regulatory effluent compliance requirements contained in the Environment Act Licence No. 2716R, while the expansion is in response to anticipated population growth in the SEWPCC service area. A primary reason for considering current and emerging regulatory compliance trends directed at treated wastewater is to assess their influence on the sustainability of a chosen treatment process design. Sustainability is often referred to as the “triple bottom line” and consists of the three major elements of: safeguarding the environment from harm; protecting and fostering the public’s interests/well-being; and achieving least cost whole-life solutions. The challenge is to find treatment solutions that fulfill current compliance requirements while being flexible enough to upgrade to meet more stringent regulatory requirements in the future all within a sustainability framework. This section summarizes some of the key current and future regulatory trends imperative to review during development of upgrade / expansion plans for the SEWPCC.

Potential changes to future plant effluent compliance criteria are discussed based upon recent trends in the wastewater industry. In general, the trend is toward lower effluent concentrations for specific constituents along with increased compliance monitoring and reporting. Prudence requires consideration of possible future effluent criteria when designing a new facility. Potential retrofits or modifications are more economically achieved when actively planned in advance. In extreme cases, the anticipated adoption of future effluent criteria can affect process selection. Emerging trends in the wastewater treatment field have potential to impact the expansion/upgrade of the SEWPCC, in particular for adherence to lower effluent limits and their associated averaging periods for with ammonia, total nitrogen, total phosphorus, and total suspended solids. There is also potential for future limits to be set for micro-pollutants and emerging contaminants of possible concern, e.g., pharmaceutical, personal care products, and surfactant residuals present in wastewater effluents.

The general approach used to assess the implications of emerging regulatory trends on the design requirements for the SEWPCC involved two key areas of focus:

- Identification of regulatory/compliance trends
- Potential implication and possible mitigation

3.2 REGULATORY FRAMEWORKS / ISSUES

The operation of the SEWPCC to achieve specific effluent limits currently involves a number of regulatory/legislative issues which includes but not limited to the following:

- *Fisheries Act (federal)*
- *Manitoba Environment Act (provincial)*

- *Canadian Council of Ministers of the Environment (CCME) (federal)*
  - Canadian Water Quality Guidelines for the Protection of Aquatic Life
- *Environment Canada (Wastewater Systems Effluent Regulations)*
- *Lake Winnipeg Action Plan (provincial)*
- *Manitoba Surface Water Quality Standards Objectives, and Guidelines*
- *The Public Health Act (provincial)*
- *The Water Protection Act C26 (provincial)*
- *US EPA – Clean Water Act, and National Pollutant Discharge Elimination System*
- *The Canadian Environmental Protection Act, 1999 (CEPA 1999)*
- *Bill 46, The Save Lake Winnipeg Act (applies to NEWPCC only)*

The above are all considerations in the effluent conditions, and are contained in some capacity in the Manitoba Environment Act Licence #2716R (revised June 19, 2009) for the SEWPCC.

3.3 CURRENT EFFLUENT CRITIERA

The Program Team provided specific effluent criteria, as presented in Table 3.1. The definition/validation phase of the SEWPCC Expansion/Upgrade design was based on these tabulated values.

Table 3.1 – Effluent Criteria

Parameter	Averaging Period	Limit	Units
Total Suspended Solids (TSS)	30-day rolling average	≤25.0[a]	mg/L
5-day Carbonaceous Biochemical Oxygen Demand ($cBOD_5$)		≤25.0[a]	mg/L
Total Phosphorus (TP)		≤1.0[a]	mg/L
Total Nitrogen (TN)		≤15.0[a]	mg/L
Ammonia Nitrogen - January	Daily never-to-exceed	≤1,975[a]	kg/day as N
Ammonia Nitrogen - February		≤2,403[a]	kg/day as N
Ammonia Nitrogen - March		≤4,196[a]	kg/day as N
Ammonia Nitrogen - April		≤12,926[a]	kg/day as N
Ammonia Nitrogen - May		≤5,311[a]	kg/day as N
Ammonia Nitrogen - June		≤3,103[a]	kg/day as N
Ammonia Nitrogen - July		≤1,517[a]	kg/day as N
Ammonia Nitrogen - August		≤607[a]	kg/day as N
Ammonia Nitrogen - September		≤703[a]	kg/day as N
Ammonia Nitrogen - October		≤811[a]	kg/day as N
Ammonia Nitrogen - November		≤1,152[a]	kg/day as N
Ammonia Nitrogen - December		≤1,550[a]	kg/day as N
Ammonia Nitrogen - Year-round: Lethal to fish	Never-to-exceed	≤50%[b]	fish mortality
E-coli and Fecal coliform	30-day geometric mean	≤200.0	MPN/100 mL

**Notes:* *a – 24 hour effluent composite sample*
b – 96 hour static acute lethality test, pH adjusted

Note: On August 2, 2011 a draft licence was issued to the City of Winnipeg for review and comment. Manitoba Conservation removed the requirement associated with fecal coliform monitoring and reporting.

3.4 FUTURE REGULATORY TRENDS FOR TARGET PARAMETERS

3.4.1 $cBOD_5$ and TSS

The effluent compliance requirement of 25 mg/L on a 30-day rolling average for both 5-day carbonaceous biochemical oxygen ($cBOD_5$) and total suspended solids (TSS) is currently under review. Manitoba Conservation on August 2, 2011 proposed revising the TSS and $cBOD_5$ compliance requirements to a not-to-exceed 25 mg/L, 98 percent of the time basis. While it is possible to meet this requirement, it is a much more stringent requirement and very sensitive to the frequency and magnitude of wet weather loading the plant might experience in the future. This will require the plant to be operated such that it provides consistently high effluent quality year-round, and ignores periods when the receiving environment has excess assimilative capacity to safely accept additional effluent loads, e.g., spring freshet. A “98%” compliance limit would result in a plant that will have to be oversized to capture infrequent excursions. To meet a never-to-exceed 25 mg/L limit, and to achieve compliance with other parameters, it will require the overall treatment process to be designed to achieve an effluent quality in the 10 to 12 mg/L range or less for both TSS and $cBOD_5$, possibly single digit values, so that intermittent wet weather loading do not cause effluent TSS to exceed 25 mg/L more than 7 days per year. The validation of the design is based on achieving a 30-day rolling average of 25 mg/L for both TSS and $cBOD_5$. Based on the frequency and magnitude of influent TSS and $cBOD_5$ loading associated with spring thaw and rainfall events, the clarification processes and biological treatment processes will need to be increased in size and performance should the Regulator impose a 98 percent compliance of 25 mg/L for both TSS and $cBOD_5$. This requirement will trigger the need to have additional discussion with potential HRC and BAF suppliers, to inform them of this performance requirement and have the unit operations and processes sized accordingly.

Since the plant upgrade and expansion is designed for projected flows and loads associated with the year 2031, it will have excess capacity until the projected flows and loads are reached. As such, it is anticipated that the removal performance of the SEWPCC will be greater in its early years of operations and allow dialogue with the Regulator to determine the most appropriate requirements on the compliance limits and averaging periods associated with TSS and $cBOD_5$.

The proposed change by Manitoba Conservation to require both TSS and $cBOD_5$ to be within 25 mg/L 98% of the time rather than on a 30-day rolling average basis will require the biological and clarification treatment process to be enlarged. Previous analysis by Stantec found that in order to meet a never-to-exceed limit of 25 mg/L for a design population of 230,000 people would require about $60 million worth of additional capital works associated with treatment processes. The current design population of 270,000 for the year 2031 will place a greater TSS and $cBOD_5$ load on the SEWPCC, which have been considered in design of the SEWPCC for flows up to 325 ML/d. The main concern relates to flows greater than 325 ML/d resulting from snowmelt or wet weather events. Based on National Climate Data records assembled by Environment Canada for Winnipeg as measured at Winnipeg Richardson International Airport (Canadian Climate Norms 1971-2000), rainfall on average has been characterized as noted below:

- 76.9 days with rainfall $\geq$ 0.2 mm
- 23.3 days with rainfall $\geq$ 5.0 mm
- 12.5 days with rainfall $\geq$ 10.0 mm
- 2.9 days with rainfall $\geq$ 25.0 mm

A Combined Sewer Overflow Strategy Study (Wardrop/Tetres, 2002) estimated that combined sewer overflows would occur when runoff exceed the equivalent of 3.0 mm of rainfall. On average, anticipated moisture conditions and depression storage would extract about 2.0 mm of rainfall. As such, 5.0 mm would be sufficient to generate about 3.0 mm of runoff and associated extraneous inflows into the combined sewer system and result in overflows to the local rivers and additional wet weather flows to the wastewater treatment plants. It is important to note that the days of rainfall are long-term averages and as such, for any given year there will be a chance it will be greater 50% of the time. As such, to achieve 98% compliance, which is equivalent to meeting the TSS and $cBOD_5$ requirement 358 days per year, or a permissible 7 excursion days per year is dependent on the rainfall in any given year. To properly assess the treatment system performance, the projected flows and associated loadings analyses will need to be expanded to include all the historic data in order to confidently determine the design required to comply with the proposed 98% compliance requirement. As an approximation for budgeting purposes, the design should consider compliance with projected maximum day conditions and be reviewed and refined in subsequent design phases, including the possible implications of climate change since it may result in more precipitation than experienced in the past.

As noted in the Administration Report “South End Water Pollution Control Centre (SEWPCC) Design Criteria” submitted by the Water and Waste Department on January 26, 2011 *“Previous river water quality investigations determined that the local rivers have robust assimilative capacity under low flow conditions and that reducing the load for either ammonia or $cBOD_5$ beyond a 30-day rolling or monthly average provides no measureable environmental benefit”.*

3.4.2 Licence Limits for Ammonia-N

Stantec recently submitted a report to the City titled “Application of the New US EPA 2009 Regulation on Ammonia to the SEWPCC”. A majority of the following discussion is based on this report. Previous river water quality investigations determined that the local rivers have robust assimilative capacity under low flow conditions and that reducing the load for either ammonia or $cBOD_5$ beyond a 30-day rolling or monthly average provides no measurable environmental benefit.

Manitoba's objectives for ammonia in the Red and Assiniboine rivers have varied substantially over the past 20 years. They have generally been based on the US EPA criteria which has increased and decreased in stringency over the years, depending on the current available science. The US EPA updated the ammonia objectives for receiving waters in 2009 and Manitoba Water Stewardship could potentially adopt these as Manitoba Objectives. The applicability of the criteria is founded on the basis that one of the two mussel species used in the 2009 US EPA criteria, the *lampsilis silihe quoidea* (Fatmucket), is found in Manitoba. However there is a great deal of uncertainty as to whether the criteria is appropriate to the local rivers, and this has a direct impact on decisions that affect design and upgrade cost to wastewater plants.

The application of the US EPA 2009 ammonia criteria in Manitoba and subsequently to the SEWPCC licence would lead to significantly lower effluent ammonia load limits. The implication would be that effluent concentrations on a monthly average, or more likely as a daily limit, would be significantly less than 1.0 mg/L in the lowest month. Manitoba Conservation has historically used the Chronic (30 Day Limit) as Daily Never-to-Exceed Limit. A more reasonable and pragmatic approach would be to develop a licence with two columns in the table for compliance assessment purposes as follows:

- Use the chronic (30-day average) load limit as the average monthly limit
- Use the acute (24-hr average) load limit as the daily limit

Applying the criteria, consistent with the approach currently taken by the Provincial regulator, the worst case scenario is presented in Table 3.2. Meeting these new and lower ammonia objectives for any secondary process designed to remove nitrogen can be extremely challenging and possibly beyond the reliability limits of current technology. As such, an important design consideration associated with BAF technology is its ability and configuration (i.e., single stage nitrification-denitrification, or separate stages for nitrification and denitrification) to reliably achieve lower effluent limits as noted in Table 3.2.

Table 3.2 – Worst Case Interpretation of Future Allowable Effluent Ammonia Load Limits - Chronic Monthly Load Limit (Applied as Daily Limit)

Month	US EPA 2009 Ammonia Load Limit (kg/d)	Existing Licence Daily Maximum Ammonia Load Limit (kg/d)
January	290	1,975
February	366	2,403
March	682	4,196
April	2,517	12,926
May	690	5,311
June	228	3,103
July	143	1,517
August	54	607
September	92	713
October	195	811
November	178	1,152
December	256	1,550

Source: Application of the New US EPA 2009 Regulation on Ammonia to the SEWPCC (September 2011) – A report by Stantec

Currently, the annual average ammonia effluent load is about 1550 kg/d-N, and expected to increase to about 2300 kg/d-N in the year 2031. For the month of August, this will require the treatment technology to have a dry weather removal efficiency of 96.4 % at present, and 97.6% in 2031. Based on an average influent ammonia concentration of 25 mg/L-N, this anticipated requirement will require the effluent concentration to be less than 0.8 mg/L-N now, and less than 0.6 mg/L-N in 2031. The influence of cold wastewater temperatures at these low concentrations will be an important factor the suppliers will need to consider in the sizing and design of their respective BAF technology to meet these low ammonia limits. This will require the near complete oxidation of ammonia, as well as the ammonia generated from the decay and mineralization of organic nitrogen, as part of the BAF treatment process. The blending of flows will need to be reviewed to determine the maximum flow that can receive only primary treatment and be blended with BAF treated effluent and still meet the final effluent ammonia limits. As well, the treatment of centrate and the residual ammonia load recirculated from this process to the main stream process will be an additional factor to consider in meeting these low ammonia limits. Since BAF treatment is proprietary, this information will be unique to manufacturer supplying the technology. It is recommended that BAF manufacturers interested in the supply of their technology provide information on the maximum sustainable removal rates, especially for cold wastewater temperatures at or below 8°C.

It is uncertain how and when the Regulator may apply the new effluent ammonia load limits. Nonetheless, the updated US EPA ammonia limits are lower and will likely be reflected in future licence compliance requirements. As such, it is prudent to review the capacity limits of the BAF treatment technology selected to achieve compliance with ammonia effluent limits given in Table 3.2.

To meet these low ammonia limits will require the near complete oxidation of ammonia, which is most effectively achieved by separate processes for nitrification (i.e., ammonia to nitrate) and denitrification (i.e., nitrate to nitrogen gas). Since the conversion of ammonia to nitrate is a function of oxidation, provided there are no other limitations (e.g., sufficient oxygen and alkalinity, pH >7, etc.), it is the oxidation process associated with the fixed film process that ultimately controls how much ammonia can be biologically transformed. As such, discussions with BAF vendors will be required to address possible staged configurations of BAF nitrification and denitrification processes, and their associated performance capacities. For example, it is possible to have a dedicated nitrification stage sized to reach low ammonia concentrations followed by a carbon assisted denitrification stage to achieve the desired total nitrogen within the limits of currently available technology, or an initial simultaneous nitrification-denitrification (SDND) stage followed by separate sequential trimming stages of supplemental nitrification and carbon assisted denitrification.

The proposed processes can be expanded and do not appear to limit the addition of supplemental treatment technologies to meet lower ammonia limits in the future.

3.4.3 Total Nitrogen (TN) and Total Phosphorus (TP)

Driven by the Provincial Water Protection Act C26 as well as influences from other Canadian and US regulations and Acts, the general regulatory trend is toward more stringent and reduced effluent limits for TN and TP. More rigorous application of regulatory licencing or permitting is applied in areas where population growth, industrialization, and agricultural activity represent a significant portion of the nutrient assimilative capacity of a receiving water body. The effluent limits of 15 mg/L as total nitrogen and 1 mg/L as total phosphorus are readily achievable and sustainable based the capabilities of current treatment technologies.

Many treatment plants in the United States are or have been designed to achieve 3 mg/L TN and TP ≤ 0.1 mg/L on an average annual basis to protect sensitive receiving waters or where water reuse is practiced. Recently, a detailed review was conducted by the Water Environment Research Foundation (WERF) to assess the reliability of these plants to attain these limits on a consistent basis. The results indicated a high degree of variation related to the choice of technology and loading characteristics on the plant. North American treatment technology has typically favored an activated sludge biological nutrient removal (AS BNR) for the removal of both nitrogen and phosphorus, (e.g., Modified Johannesburg and Westbank processes) based on their ability to achieve low TN and TP limits biologically. In most cases, tertiary treatment is required to reduce the suspended solids to achieve the TP limits.

The application of BAF technology is becoming more common in North America due to its ability to achieve low TN levels cost-effectively, smaller footprint, and flexibility to integrate with chemical and/or biological phosphorus removal processes. At present, to achieve low TN involves near complete nitrification of ammonia followed by denitrification with a supplemental carbon source such as methanol. As such, the implementation of BAF technology will need to consider flexible arrangement of nitrification and denitrification processes as part of the treatment design and configuration of unit operations so that it can be transitioned in the future as required to meet lower TN limits.

It is anticipated that Manitoba will lower the limits to $\leq$ 10 mg/L for TN and $\leq$ 0.3 mg/L for TP. The adoption of these lower effluent nutrient limits in the future could be addressed via process modifications and/or the addition of effluent filtration to the process. For instance, effluent phosphorus limits of 0.3 mg/L could be achieved with granular filtration.

A cursory review was done as part of Section 7 – Concept Development to provision space and strategically local equipment and process to facilitate conversion to a biological based phosphorus removal system. To reliably meet TP limits down to 0.3 mg/L it is recommended that a chemical trimming system be maintained to add metal salt as required to achieve the desired TP limits. To achieve TP limits below 0.3 mg/L will likely require some form of effluent filtration to achieve low effluent TSS concentrations in order to capture the phosphorus contained in the particulate matter. The efficiency of phosphorus removal treatment systems will need to be validated in full-scale to more accurately determine if TP concentration of 0.3 mg/L can be achieved without filtration.

Phosphorus recovery is primarily accomplished by the removal of particulate phosphorus by settling or filtration, and by the formation of a struvite like granule. The use of iron (Fe) and aluminum (Al) based metal salts creates an inert substance that is not readily bioavailable to plants. As such, to maximize the recovery of bioavailable phosphorus will require the minimization of the use Fe and Al metal salts in the main stream, or conversely the maximization of biological phosphorus removal. Solids removed from the main stream processes typically undergoes some form of digestion to stabilize the residual solids. Digestion normally releases high concentrations of ammonia and soluble phosphorus. The dewatering process will concentrate the ammonia and soluble phosphorus in the reject water, which makes it favorable to the controlled formation of struvite. Specialized processes have been developed that include the addition of magnesium (Mg) along with pH adjustment to encourage the controlled formation of struvite granules. Should biological phosphorus removal and digestion be implemented at the SEWPCC, it is recommended that options for phosphorus recovery from centrate be investigated to allow for its coordination integration with the overall site development.

3.4.4 Biological Phosphorus Removal

The existing SEWPCC licence does not stipulate the process by which phosphorus is to be removed. However, unlike the SEWPCC licence, the Save Lake Winnipeg Act stipulates that "*Nutrient removal must be achieved primarily by biological methods through application of the* *best available biological nutrient removal technologies. Nutrients that are removed must be recovered and recycled to the maximum extent possible through application of the best available technologies*." at the NEWPCC. Based on this precedence, there is a strong likelihood that biological P removal at the SEWPCC will be legislated along with recovery of P from its residual solids.

It is anticipated that Manitoba Conservation will mandate biological phosphorus removal (BPR) in the near future with the intent that residual P in the sludge be recovered for beneficial reuse. As such, it is recommended that provision be made in the design for the conversion to biological P removal and its recovery in the future.

3.4.5 Nitrate

The CCME has announced that they are reviewing effluent nitrate concentrations because of its potential negative impacts on aquatic life. Currently there are no effluent limits for nitrate unless a drinking water intake is immediately downstream of a wastewater treatment plant discharge. The current recommended limit for nitrate (NO_3) in freshwater is 13 mg/L as NO_3, this translates to 2.9 mg/L as N. (Environment Canada, 2003, Canadian Water Quality Guidelines for the Protection of Aquatic Life: Nitrate Ion, Report No. 1-6).

To achieve low levels of ammonia requires a high degree of nitrification, and to achieve low levels of nitrate requires the additional step of denitrification. Achieving low limits for both ammonia and nitrates currently favors separate unit processes and often requires the addition of alkalinity to maintain nitrification and supplemental external carbon source for denitrification. The requirement for low nitrate levels requires near complete denitrification and can incur high operating costs associated with electricity (oxygen supply), and chemical supply (alkalinity and supplemental carbon). The requirement to achieve lower effluent ammonia concentrations will increase the amount of nitrates generated as part of the treatment processes. The increased amount of nitrates will constitute a greater proportion of the total effluent nitrogen concentration. It is recommended that provision be made in the design to expand the denitrification capability to meet these low nitrate limits, and that a conservative approach be used to estimate the methanol required and sludge generated to achieve a TN of 5 mg/L-N, and for this expanded capability to be assessed as part of the preliminary design.

3.4.6 Emerging Contaminants of Potential Concern

The increased use of pharmaceuticals, personal care products, and other new synthetic compounds have resulted in trace amounts of these substances in wastewater, and these are making their way into the environment through effluent discharges. The chemical, physical, and microbiological processes used in wastewater treatment plants are being studied to help understand how much, if any, of these substances are being removed prior to discharge. Recent information indicates that longer solids retention times associated with activated sludge nitrification systems, and plants with sludge digestion, do have a positive but limited removal benefit on certain substances. Specific technologies such as Reverse Osmosis (RO) can remove a significant fraction of these substances but is often limited to applications where water reuse is required due to its high cost. Ongoing scientific research is gathering information to better understand the long-term impacts of emerging contaminants of potential concern (e.g., endocrine disrupting chemicals – EDCs on fish). This information will help determine their fate in the environment impact on aquatic life, impact on public health, substances that need to be controlled, and how best to accomplish this challenge. Current efforts are focused on source controls where practicable. The timing and need to removal of emerging containments of concern is highly uncertain at present and their consideration premature in the selection of technology as part of the SEWPCC upgrade and expansion.

4.0 Design Flows and Loads Validation

4.1 INTRODUCTION

Flows and loads were updated for the SEWPCC upgrade / expansion based on more recent and complete information for wastewater characteristics, and forecasts for population growth in the service area revised by the City's economist – G. Chartier. Since wastewater loads are linked directly to population, recent adjustment of population projection from 230,000 to 270,000 people will result in greater loads to the treatment plant for the selected design horizon of 2031. However, new developments are expected to generate significantly less wet weather-induced extraneous inflows since weeping tile flows are no longer directed to the wastewater sewer and better methods and materials are used in the design and construction of sewers. To allow some extraneous wet weather flow contribution from new areas, the Program Team suggested that an allowance for 50 percent of wet weather flows for existing areas would be used for new developments. The general approach used to develop the design flows and loads was as follows:

- Update current and future projected dry and wet weather flows
  - The primary purpose of projecting dry and wet weather flows to the SEWPCC is for assessment of hydraulic capacity and design during conditions of normal plant operation under dry and wet weather conditions (i.e., within primary treatment capacity) and for flood protection requirements associated with extreme snowmelt or rainfall conditions and high river water levels (i.e., total and firm pumping and conveyance capacity). A review was conducted on the available hourly flow data from January 2005 to June 2011, as received and recorded at the SEWPCC. The wet weather induced flows were scaled up to estimate the amount spilled to the river in response to emergency sanitary sewer overflows. This would provide a better system-wide estimate of existing extraneous flow contributions for the developed area contained within the service boundary of the SEWPCC. Dry weather flows are directly related to sector composition and population. Based on development plans as presented in Our Winnipeg – Complete Communities (July 12, 2011) the sector composition for the SEWPCC service area is not planned or expected to change significantly from its current distribution. As such, it is expected that current wastewater load generation trends will be representative of future trends.
  - The focus on sustainable neighborhoods creates emphasis on conservation of land and the densification of population.
  - It is noteworthy that water consumption models predicted an average per capita usage of 169 litres per day in the year 2031. A review of trends indicates that the existing service area population has implemented newer and more efficient water devices at a pace faster than originally anticipated. This positive downward trend has realized a lower

water consumption rate earlier than expected and this trend is not expected to change significantly in the future. With these factors considered, it is expected that loads to the SEWPCC will increase in direct proportion to population growth but dry weather concentrations will be higher due to reduced wastewater flow generation related to lower water consumption.

- Wet weather flows will increase, but it is difficult to predict the extent of such increases with accuracy due to the possible variability associated with climate changes, deterioration and maintenance issues within the existing sewer system, possible separation of combined sewer systems; potential illegal connection of sump pumps to private sewer services; and sewer conveyance system upgrades. For this report, it was assumed that no major upgrades to the interceptor system would take place prior to 2031. As such, the wet weather flows reaching the SEWPCC are limited to the current hydraulic capacity of the collection and conveyance system.

• Update current and future projected seasonal loads for specific wastewater constituents

  - The primary objective of this task is to estimate projected future loads based on population growth and an assessment of recent wastewater characteristics. For purposes of consistency, the analyses were done in a format that permitted the direct comparison of factors and loads contained in the Process Selection Report (Veolia, August 2011). As noted earlier, it is expected that the sector composition for the SEWPCC service area will remain largely unchanged for the design horizons of 2031 and 2061. For design purposes, the 98^{th} percentile of the existing loads for specific parameters was selected. The loads were then directly scaled up based on populations of 270,000 for the year 2031, and 400,000 for the year 2061. The loads estimated from this exercise formed the basis for the sizing of unit operations and treatment processes.

• Develop a stress pattern to test the performance and compliance of plant designs based on future flows and loads.

  - An internal independent review was performed on the wastewater flow and quality data for specific constituents provided by the City of Winnipeg, Water and Waste Department. The objectives of this exercise were to independently validate both the flow and load assessments for current and projected populations were within the expected ranges, and to assess compliance with effluent criteria provided by the Program Team for specific performance efficiencies associated with the treatment components (i.e., CEPT, BAF and HRC) and various flow splits. Specifically, the flows and loads from the previous exercises would be used for design purposes; while the stress pattern would be used to test the ability of various design schemes to achieve compliance with the effluent criteria provided by the Program Team.

This section describes the technical analyses associated with flows and loads relevant to the process and hydraulic design of the SEWPCC upgrade and expansion. It is important to note that the analyses evolved as the assignment proceeded, that is, each aspect was done in sequence. The analyses and results were refined progressively and further validated as work progressed. Accordingly, the stress pattern was developed last and is a key product as it is considered to be the most complete and comprehensive compilation of flows and loads.

4.2 DEVELOPMENT OF DESIGN FLOWS

4.2.1 Flow Technical Analyses

Estimates were developed for future wastewater flows to the SEWPCC for a 20- and 50-year horizon in order to assist in the hydraulic design requirements associated with the upgrade and expansion of the treatment facility. The 20-year horizon corresponds to the year 2031, and the 50-year horizon corresponds to the year 2061. The projected flows also served as the foundation for estimating future loads to the SEWPCC for design purposes. These flows and loads will be used to approximate the sizes of the unit processes and operations to achieve compliance with effluent concentrations and loads for specific parameters as provided by the Project Team (i.e., Veolia and the City of Winnipeg). It is important to note that the SEWPCC under certain environmental conditions, is a vital part of the overall flood defense system protecting homes and business from widespread flooding. Due to the complexity of the gravity sewer system and the operation of major lift stations, limitations where imposed on the peak flows that could be conveyed to the SEWPCC for the 20-year horizon. Conversely, unconstrained conditions were assumed for the 50-year horizon and serve as an indicator of the expansion required in the collection system to facilitate unconstrained development in the existing, and future expanded service area.

The following SEWPCC data was provided by the City of Winnipeg Water and Waste department for use in flows and load analysis:

- Daily flow and quality data (January 1, 2005 to June 20, 2011)
  - Maximum, minimum, and average daily flow (ML/d)
  - Raw influent, primary effluent, and final effluent
    - Temperature (°C)
    - pH
    - Alkalinity
    - TSS
    - BOD
    - $cBOD_5$
    - TOC
    - Filtrate TOC (SOC)
    - COD

- Total VFA
- TKN
- Total Ammonia
- Nitrite plus Nitrate
- Ortho-P
- Total P

- Diurnal flow (ML/d)
  - 2005 to 2009, in 6 minute increments
  - 2010 to July 2011, in 1 minute increments
- Population projections by the City economist – G. Chartier (based on 2009 census data)
- Administrative Report to Council (January 26, 2011) recommending the use of a service area population of 270,000 for the design horizon of 2031

The generalized approach used to estimate flows and loads to the SEWPCC is shown in Figure 4.1. The approach used to derive dry and wet weather flows to the year 2031 based on current and projected populations is illustrated in Figure 4.2. The following sections will provide more detail on how these approaches were applied to derive flows for hydraulic related design reasons, and later used to develop loads for treatment process design reasons.

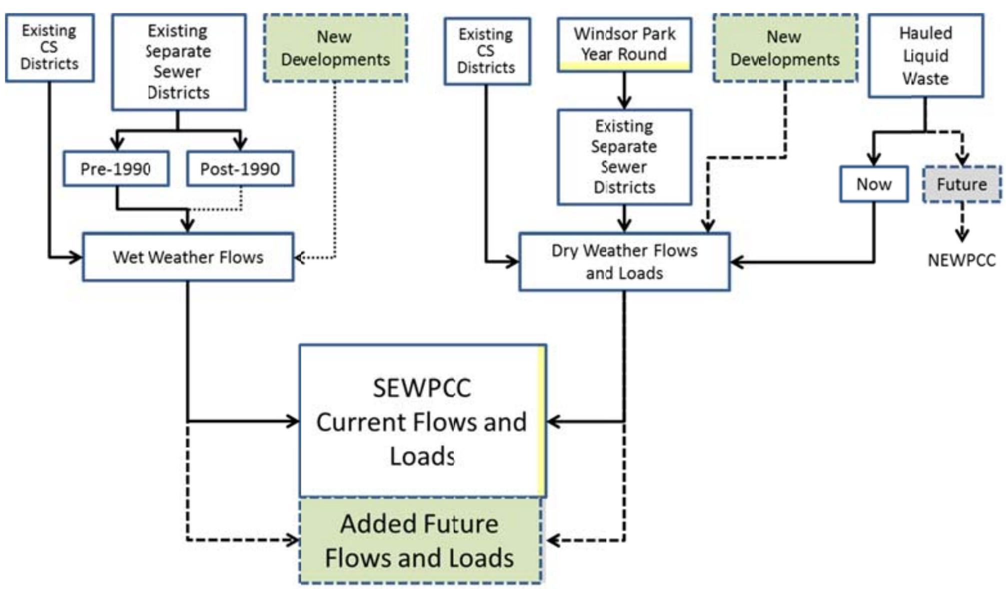


Figure 4.1: Generalized Approach to Estimating Flows and Loads to SEWPCC

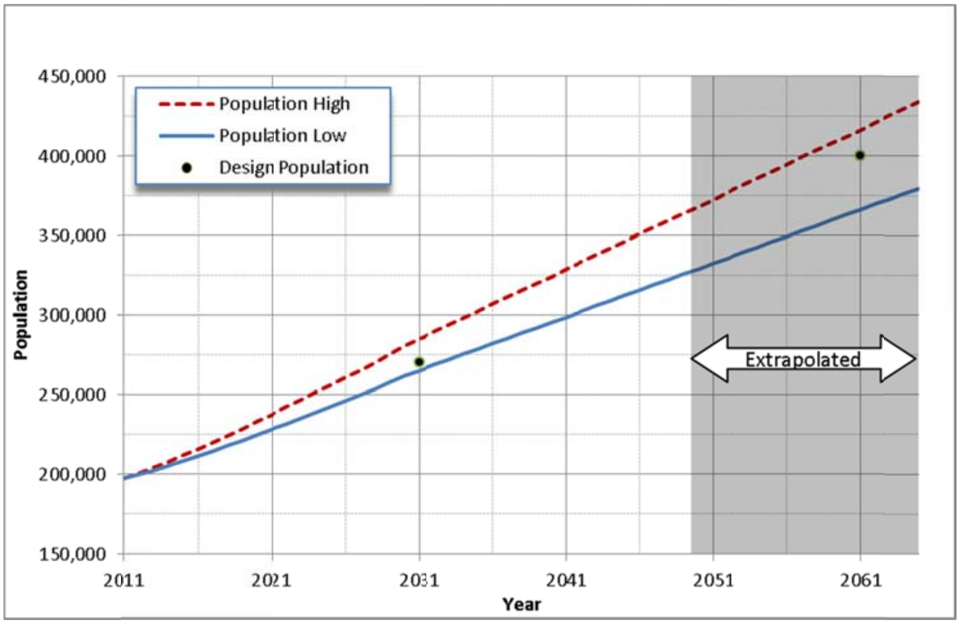


Figure 4.2: Projected SEWPCC Population[1]

In general, extraneous wet weather flows originating from rainfall or snowmelt conditions can enter the wastewater sewer system via the following routes:

- Directly via catch basins found in combined sewer (CS) districts
- Directly via weeping tiles for homes built prior to 1990
- At manhole entry ports (e.g., seepage and leakage through lift holes and rim)
- Through groundwater leaks around manhole frame and cover, barrel sections, and at pipe joints
- At cross-connections with land drainage systems

Homes built after 1990 were required to have sump pumps that discharged the collected foundation drainage to the street surface to explicitly remove this extraneous inflow from the wastewater sewer system. More stringent development standards, construction methods, and materials further reduce the possible entry of extraneous wet weather induced flows into the wastewater sewer system. These improvements are intended to reduce extraneous wet weather flows entering the wastewater sewer system from new developments and support a less extraneous inflow contribution.

4.2.2 Population Projections

The SEWPCC service area is experiencing a rapid increase in population. The City has projected that the 2031 service area population for the SEWPCC would range between 265,000

[1] Data source: City of Winnipeg Economist – G . Chartier (up to 2048)

to 285,000 people, as shown in Figure 4.2. In a report to City Council (Council Minutes – February 23, 2011), a 2031 population of 270,000 was adopted for the design of the SEWPCC upgrade and expansion, and represents a growth factor of about 1.5% per year. Based on an extrapolation of the population growth information provided by the City, and by applying the same growth factor of 1.5% per year to the year 2061, the projected population for the SEWPCC is expected to be in the range of 366,000 to 416,000. As such, a service area population of 400,000 was adopted for long-range planning purposes, and forms the basis for long-range development plans for staged and modular plant expansion of the SEWPCC facilities.

4.2.3 Dry Weather Per Capita Wastewater Generation Rate

An assessment was done on the current per capita wastewater generation rates to estimate the dry weather flow to SEWPCC and also to predict future flows. It was assumed that the sector composition in the SEWPCC service area would remain proportionally the same and be representative of future conditions.

Dry weather flow (DWF) is defined as the base flows to the wastewater treatment plant during the winter months of December, January and February when the amount of extraneous wet weather flows are expected to be at a minimum. The historic DWF received at the SEWPCC is shown in Figure 4.3. The average dry weather flow was calculated to be 48 ML/d and the average annual flow 63 ML/d, based on a review of the last six years of flow records (2005 to 2010, inclusive) as measured at the SEWPCC.

The corresponding mean population for this period was 184,000 people, yielding an average per capita wastewater generation rate of 260 liters per capita per day (L/c/d). The current per capita wastewater generation rate was found to be lower than previously estimated (Stantec, 2007). The per capita wastewater generation rate has dropped from 298 to 260 L/c/d at a much faster rate than originally anticipated. This rapid rate change warranted a review of the components that comprise the overall wastewater generation rate, as summarized in Table 4.1.

The Windsor Park sewer district has the flexibility to send its flows to either the SEWPCC or the NEWPCC. Since the NEWPCC has excess treatment capacity and there is small risk of combined sewer overflows occurring in the winter months, flows and loads from this sewer districts were purposely sent to the NEWPCC from mid-to-late October to mid-to-late April, depending on weather conditions, to minimize sludge hauling costs from the SEWPCC to the NEWPCC. For SEWPCC upgrade and expansion design purposes, the Program Team recommended that flows and associated loads from the Windsor Park sewer district be considered and routed year-round in the SEWPCC for treatment. Based on 2006 census data provided by the City's economist, (G. Chartier) the estimated population for the Windsor Sewer district is 9,665 people. Based on an average per capita wastewater generation rate between 260 and 278 L/d, the additional flow from the Windsor Park sewer district to the SEWPCC between October and April would range between 2.5 and 2.7 ML/d.

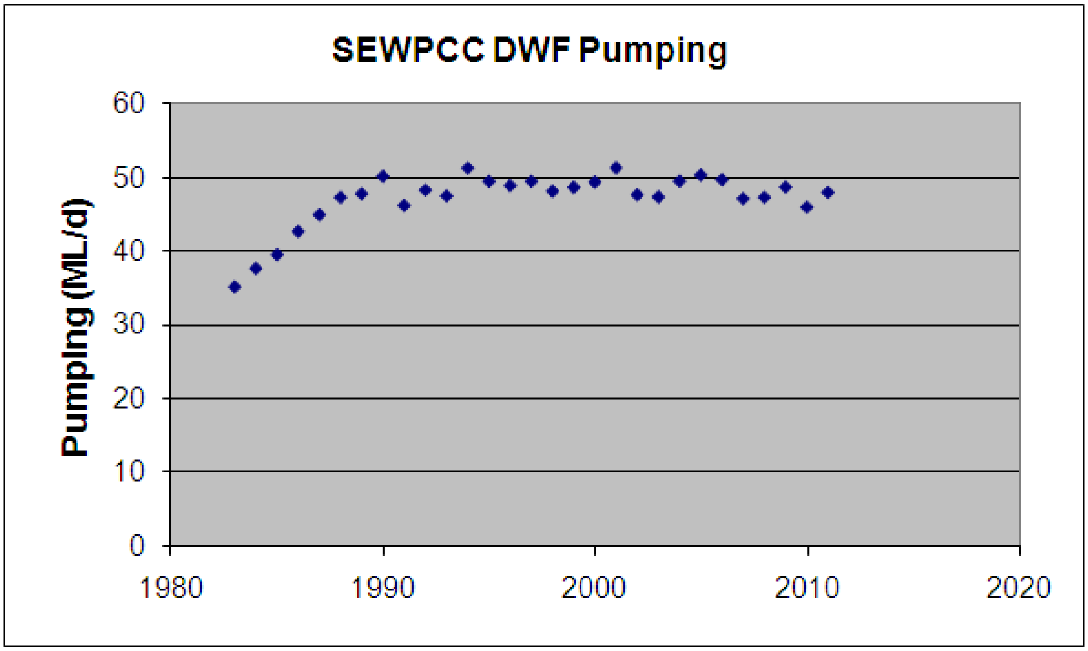


Figure 4.3: Historic Winter Dry Weather Flows to SEWPCC

Table 4.1 – Per Capita Wastewater Generation Rates (excluding Windsor Park)

Component	Stantec 2007	Current (low)	Current (high)
Residential (L/c/d)	190	170	170
Commercial (L/c/d)	45	45	45
Industrial (L/c/d)	15	15	15
Winter infiltration (L/c/d)	48	30	48
Total (L/c/d)	298	260	278
Population	229,800	270,000	270,000
Total ML/d*	68.5	70.2	75.1

* the addition of Windsor Park will add about 2.6 ML/d to the winter dry weather flows.

Based on water billing records from the City of Winnipeg, it was found that the residential wastewater component has dropped from 190 to 170 L/c/d and likely can be attributed to a more aggressive home renovation rate to more efficient water using devices (e.g., toilets, washing machines, shower heads, faucet aerators, etc.), and the use of more water efficient devices in new home construction, as shown in Figure 4.4. Based on the water consumption models that were developed by the City of Winnipeg, it is expected that future residential water usage would not change significantly and will remain stable at about 170 L/c/d. Commercial and industrial water use patterns, based on water consumption records, remained unchanged for this period.

Subtracting the residential, commercial, and industrial wastewater generation rates revealed that the groundwater infiltration was significantly less than previous years. It was uncertain what factors led to this reduction in the last six years, and whether this trend would persist in the future. Stantec recommends that the long-term winter ground water infiltration rate of 48 L/c/d be used for design reasons because it is based on a longer period of record (1983 to 2005) and would be more representative of average conditions than the last six years (2005 to 2010). Accordingly, a dry weather per capita wastewater generation of 278 L/c/d was applied to estimate the base dry flow to the SEWPCC for the year 2031. Based on a projected future population of 270,000 for the year 2031, and 400,000 for the year 2061, the inclusion of Windsor Park dry weather flows, the estimated dry weather flow would be 78 ML/d, and 115 ML/d, respectively.

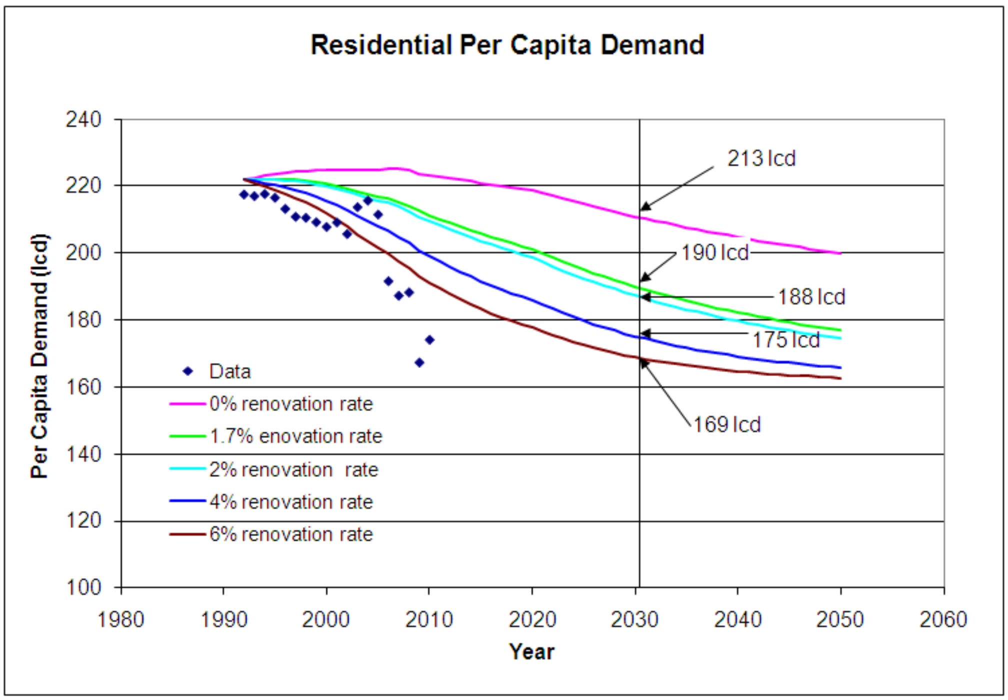


Figure 4.4: Residential Per Capita Demand

4.2.4 Peak Flows to the SEWPCC

Historic data (2005 to 2010, inclusive) was used to develop a numerical model to predict peak flows to the SEWPCC for future 2031 and 2061 conditions. Previous studies done for the City of Winnipeg on extraneous inflow and infiltration suggested that about 25% of homes in new developments might connect their sump pump discharges to their wastewater service. Enforcement and regulation is actively being done by the City to remove illegally-connected sump pumps to the wastewater sewer system. In addition, attention is being placed at placing new manhole entry ports in new developments at locations that are not prone to water ponding, and improvements are being implemented in the water tightness of lid and frame assemblies, and barrel section joints. These additional measures are explicitly intended to minimize or eliminate intrusion of extraneous flows into the wastewater sewer system. Prudence dictates an allowance for a larger amount of extraneous wet weather flows for flood protection design reasons at the SEWPCC, therefore the Program Team recommended that new areas be assumed to generate 50% of the extraneous wet weather flows generated by existing developments.

While the new developments may generate a certain peak and duration of wet weather flows in response to snowmelt or rainfall conditions, it is important to note that the existing conveyance system pumping and hydraulic carrying capacity of the interceptor systems strongly influences the duration and amount of peak flows at the SEWPCC. Since the existing wastewater pumping and conveyance system strongly influences the peak flows to the SEWPCC, it is unlikely that peak flows will be significantly greater than currently observed values but may last longer in duration. Improvements or changes to the collection and conveyance systems will strongly influence the peak wet weather flows received at the SEWPCC, and will represent a critical consideration in the flood protection and treatment requirements at the SEWPCC to comply with effluent quality criteria. For this assignment, it was assumed that no significant changes with the collection and conveyance system would be done prior to the design horizon of 2031, and this condition is referred to as “constrained” development. For long-range planning purposes, it was assumed that the existing collection and conveyance systems would be upgraded after 2031 to allow unrestricted development to accommodate population growth to the year 2061, and referred to as “unconstrained”.

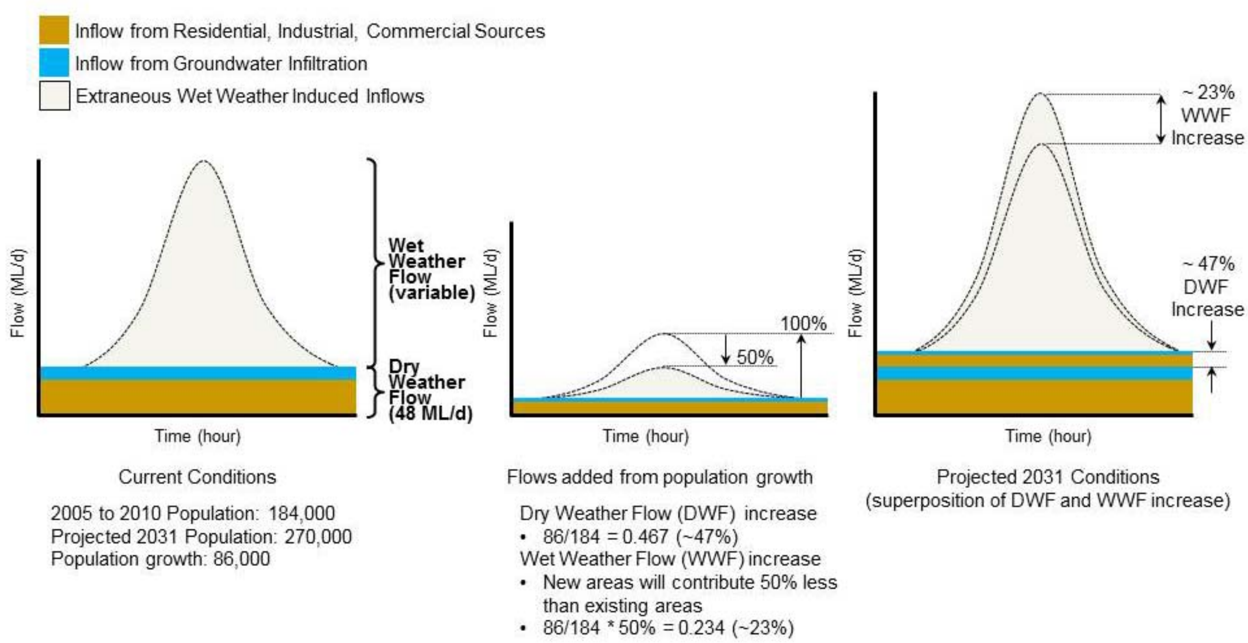


Figure 4.5: Approach Used to Estimate Projected 2031 Dry and Wet Weather Flow to SEWPCC

The diurnal patterns as well as the wet weather flows for the past six years were scaled up based on population and an allowance for extraneous wet weather flows from new developments. Based on discussions with the Program Team (Meeting notes of August 24, 2011), it was agreed that new development areas would only contribute 50% the amount of the wet weather flow generated from existing developments within the SEWPCC service area. The average population within the service area for the SEWPCC between 2005 and 2010 is about 184,000 people. Under the assumption that existing wastewater quality trends hold true for the future, the projected load to the SEWPCC will increase in direct proportion to the population, that is, by 270,000/184,000, or a factor 1.47 times the existing loads. Since new developments are assumed to be more water tight then existing developments, a lower fraction of extraneous flow will be generate in these new areas. As such, following the direction provided by the Program Team that peak wet weather flows will be 50% of that from exiting areas (equivalent to a 50% reduction in population), the projected peak flows will increase by 226/184, or a factor 1.23 times the existing peak flows. Figure 4.5 graphically illustrates the approach used to estimate future flows to the SEWPCC.

The existing south end wastewater collection and conveyance system has several emergency overflow points to protect against wide-spread basement flooding due to excessively high water levels in the sewer system. A schematic representation of the sewer system is shown in Figure 4.6.

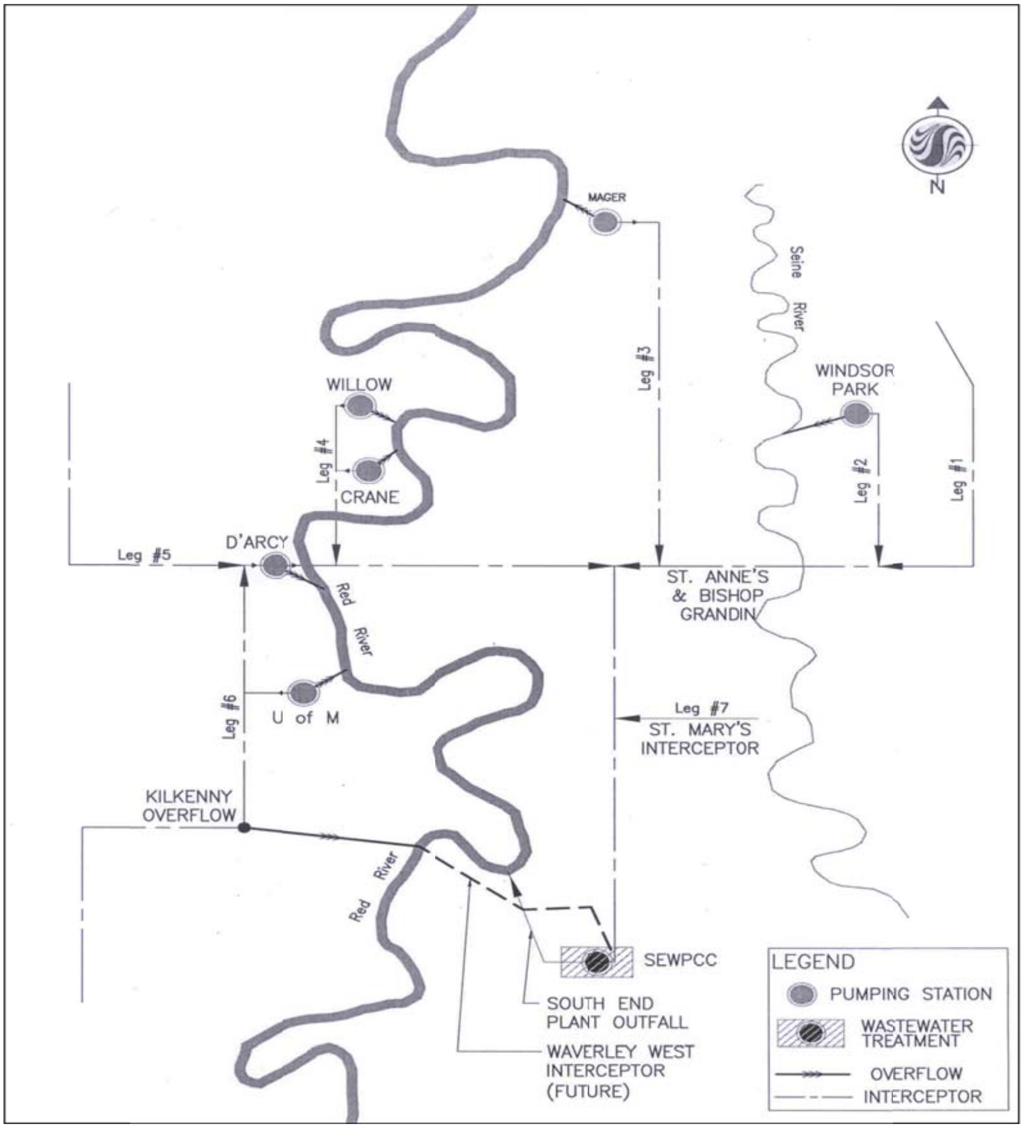


Figure 4.6: Schematic showing SEWPCC Collection System

Hydraulic analyses previously completed by Stantec (2008 Conceptual Design Report) estimated flows to the SEWPCC for a range of wet weather conditions to assess the peak flow and duration the SEWPCC might experience. The assessment at that time considered a new river crossing (shown as a dashed line) from the southwest interceptor to the St. Mary's Road interceptor, which would bring more wet weather flows at a faster rate to the SEWPCC. The river crossing was originally considered as a system optimization measure to improve overall system conveyance capacity in order to minimize wet weather overflows and improve system-wide basement flood protection. At present, the gravity flow capacity of the St. Mary's Road interceptor limits the amount and duration of peak flows that can safely reach the SEWPCC.

Previous analyses found that for the design storm used to predict the flows to the plant for the current interceptor configuration, the system could deliver a peak flow of 415 ML/d and a maximum day flow of 300 ML/d. Specifically a steeper short-term hydraulic gradient in the St. Mary's Road interceptor could convey a peak hour flow of about 415 ML/d, but this peak flow rate cannot be sustained without risk of wide-spread basement flooding due to the rising backwater in the sewer system. A steady-state peak in the range of 300 ML/d is possible at the SEWPCC with low risk of basement flooding provided that the excess flows are shed at emergency overflow locations. The upgrading of the SEWPCC outfall and proposed raw sewage pumping upgrades, in combination with proposed hydraulic conveyance systems within the plant, must be able to facilitate a sustained maximum day flow greater or equal to 300 ML/d and a peak hour greater than or equal to 415 ML/d. As such, the aforementioned flows represent the boundary conditions previously used in the peak flow design of the SEWPCC by Stantec.

Following completion of hydraulic modeling by Stantec for the design of the SEWPCC, Stantec was retained in 2011 by the City of Winnipeg, Water and Waste Department to provide engineering support services associated with flood protection and operation of the SEWPCC and real-time sewer flow forecasting. A revised hydraulic model of the sewer system was developed to provide a quick and accurate prediction of the sewer system response for observed event-specific conditions in order to allow rapid decision-making and action to protect against wide-spread basement flooding.

The knowledge and experience gained by Stantec on the SEWPCC sewer system was directly applied to predict the response and behavior of the sewer system flows and possible overflows based on the last six years of flows as measured at the SEWPCC. The previous modeling was reviewed in order to back-calculate and estimate the amount of raw wastewater released to the river as emergency overflows during large wet weather events. This information was used to develop a generalized correlation, on a system-wide basis, between wet weather events and emergency overflows. This correlation was then applied to the measured flows at the SEWPCC to estimate the amount of flow lost due to emergency overflows. An event specific (July 2005) estimate of the wet weather flows (WWF) reaching the SEWPCC and the emergency sanitary sewer overflows (SSO) is shown in Figure 4.7. The difference between the SSO and WWF lines is the volume and duration of the emergency overflows event to the Red River. Based on a projected population of 270,000 people in 2031, the flows were scaled up as noted earlier. The corresponding 2031 WWF and SSO is illustrated in Figure 4.8. It is noteworthy that the amount and duration of emergency overflows are expected to increase somewhat due to driving head, but will be throttled due to the hydraulic limitations of the existing system. If future regulations are issued to eliminate or minimize overflows to a specified limit, a review will need to be done to determine the most cost-effective and practicable methods to manage SSOs on a system-wide basis.

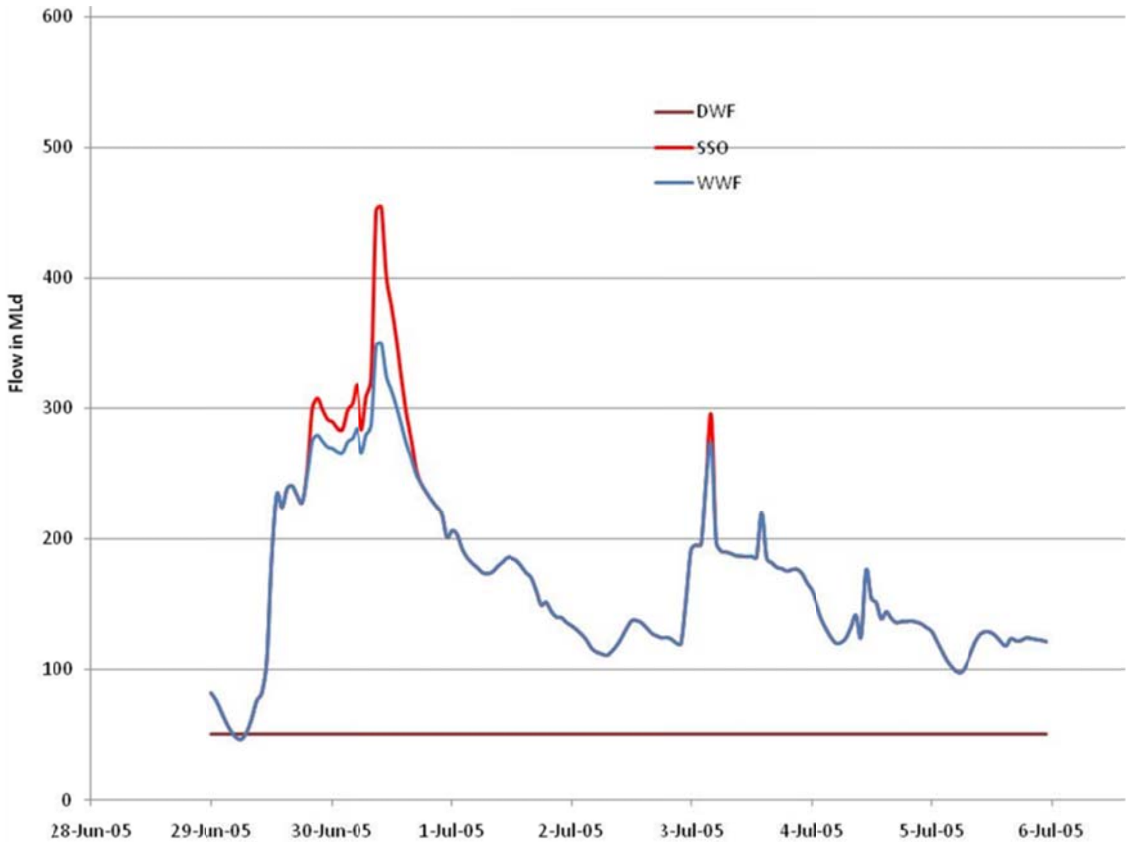


Figure 4.7: Event-specific Estimate of Emergency Sanitary Sewer Overflows (SSO) and Wet Weather Flows Received at the SEWPCC

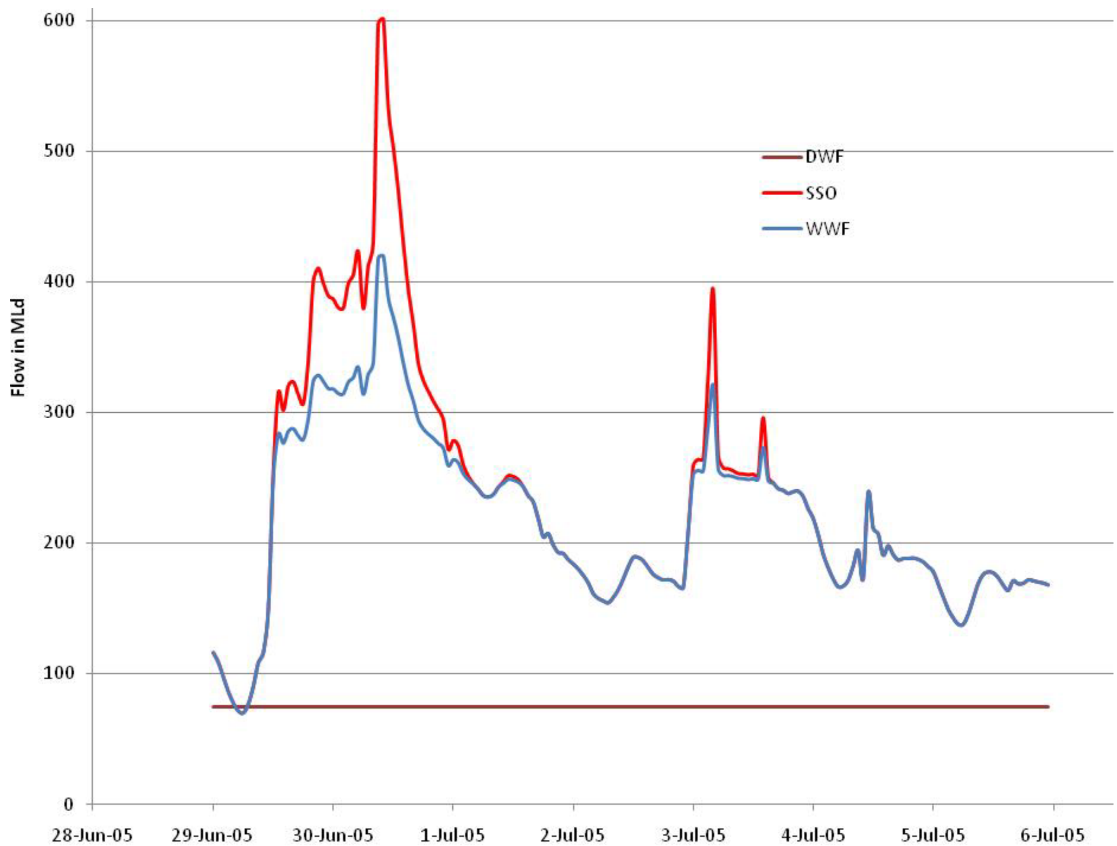


Figure 4.8: Event-specific Estimate of Emergency Sanitary Sewer Overflows (SSO) and Wet Weather Flows Received at the SEWPCC based on Projected 2031 Population of 270,000 People

In developing future flows to the plant it is recognized that the current wastewater sewer collection system and pumping stations limit the flows that can be conveyed to the SEWPCC. The approach used in the estimation of the flows conveyed to the SEWPCC via the sewer network system takes into consideration and approximates possible losses due to emergency overflows in response to excessive wet weather inflows. Based on discussions with the Program team, it was agreed that current conveyance systems performance would be used to estimate the peak flows that the SEWPCC is expected to receive in the year 2031. Conversely, it was assumed that future charges to operational regulations impacting the collections system would result in unconstrained conditions for the 50-year horizon for system-wide planning purposes. The hourly data for the years 2005 to 2010 inclusive were scaled up based on estimated populations of 270,000 and 400,000 people for the years 2031 and 2061, respectively. The hourly data for each day was parsed into the months that comprise winter, spring, summer, and fall. Statistical analysis were then completed for the seasonal data sets to calculate the peak flows, maximum day, maximum 7-day rolling average and maximum 30-day rolling averages for each season. Table 4.2 summarizes the current and project peak flows to the SEWPCC based on this approach.

Table 4.2 – Current and Project Peak Flows to the SEWPCC Based on 6 Years of SEWPCC Flow Data (2005-2010, inclusive), Actual and Revised Per Capital Wastewater Generation, and Scaled Up Based on Population Growth Plus Additional Extraneous Flows from New Developments

RECOMMENDED SEWPCC DESIGN FLOWS (ML/d)					
Year = 2005-2010	Winter	Spring	Summer	Fall	Annual
Average	48.3	70.7	71.8	59.4	63.3
Max of 30-Day Average	50.8	112	114	91	
Max of 7 Day Average	56.0	159	168	114	
Max Day	78.6	276	272	205	
Year = 2031 “unconstrained”	Max hour = 420				
Ignores hydraulic capacity limits of existing Interceptor System	Winter	Spring	Summer	Fall	Annual
Average	75.4	108	105	88.5	94.5
Max of 30-Day Average	79.8	160	174	124	
Max of 7 Day Average	85.9	210	235	154	
Max Day	114	390	402	272	
Year = 2031 “constrained”	Max hour = 420				
Existing hydraulic capacity of existing Interceptor System	Winter	Spring	Summer	Fall	Annual
Average	75.4	108	105	88.5	94.5
Max of 30-Day Average	79.8	160	174	124	
Max of 7 Day Average	85.9	201	220	154	
Max Day	114	300	324	272	
Year = 2061 “unconstrained”					
Assumes hydraulic capacity of Interceptor Upgraded	Winter	Spring	Summer	Fall	Annual
Average	112	153	149	128	136
Max of 30-Day Average	117	219	238	173	
Max of 7 Day Average	125	283	315	212	
Max Day	161	513	529	363	

The expansion and upgrade of the SEWPCC to accept and treat projected 2061 flows and loads is strongly influenced by the hydraulic capacity of the collection and conveyance systems. The existing collection and conveyance systems have sufficient capacity to convey the projected 2061 average dry weather flows but limits the amount of additional wet weather induced extraneous flows that can be safely conveyed to the SEWPCC. Emergency overflow provisions have been built into the systems operation at strategic locations in and along the collection system to shed excessive wet weather flows to protect against wide-spread basement flooding. The hydraulic capacity of the conveyance system will need to be increased to safely capture and convey additional wet weather flows to the SEWPCC. Given the regulatory trends elsewhere, there will be pressure to minimize or eliminate wet weather induced overflows in Winnipeg. The type of sewer districts (i.e., separate sewer vs. combined sewer systems), geometry and limited topography throughout the wastewater collection system likely dictates that emergency overflow provisions should be retained in some capacity as a control measure to protect against wide-spread basement flooding. The protection of public health, environmental benefits, practicality, and costs all need to be considered when determining whether to capture and covey wet weather flows to the SEWPCC or implement wet weather overflow treatment upstream of the SEWPCC. This is a major decision that warrants further assessment and planning due to the associated costs and systems implications. As part of the 2031 design the bypass piping will be designed to accommodate the 2061 flow of 680 ML/d. Prior to implementing any treatment plant expansions beyond the 2031 design horizon, Stantec recommends that a review be undertaken to identify possible future collection system upgrade/treatment options as compared to providing additional wet weather flow treatment at the SEWPCC. This review will define the long-term South End service area collection/treatment strategy.

4.3 DEVELOPMENT OF DESIGN LOADINGS

4.3.1 Loading Technical Analysis

This section summarizes the approach used to develop projected design loads. The data analysis is based on SEWPCC raw wastewater data provided by the City for from January 1, 2005 to December 31, 2010, inclusive. The data was provided electronically by the City of Winnipeg and consisted of the following data:

- Flows recorded in either 6 minute or 1 minute intervals
- Influent Temperature
- Influent pH
- Influent Alkalinity
- Influent TSS
- Influent VSS
- Influent BOD Uninhibited
- Influent CBOD Inhibited
- Influent TOC
- Influent Filtrate TOC (SOC)
- Influent COD
- Influent Total VFA
- Influent TKN
- Influent NH3
- Influent NO2/NO3
- Influent TN
- Influent orthoP
- Influent TP

Step 1 – Establish Historical Annual Average (AA) Loads

The performance of the BAF is strongly impacted by the loading placed on it; consequently the TSS, BOD_5 TKN and TP concentrations and loadings were reviewed to identify any data points that appeared to be suspect based on its comparison to the data sets as a whole (i.e., distribution). It was observed that one data point appeared to be excessively skewed and out of the expected range based on the whole data set for TSS. The TSS value recorded on May 13, 2009 was the third high concentration recorded at 504 mg/L, but its corresponding load was calculated to be 86,688 kg/d which is highest ever recorded and almost double that of the next highest TSS load value of 50,130 kg/d. As such, the data point was considered suspect and removed from all subsequent analyses. The following steps were used to estimate average annual (AA) loads.

- Historical Annual Average (AA) loadings were calculated for key design constituents (e.g., BOD_5, TSS, etc.) for the respective years.
- Based on yearly average population (obtained from Veolia PSR document dated May 2011), per cap loadings for TSS, BOD_5, TKN and TP were calculated on an AA basis for each year.
- The design per capital loadings on an AA basis was established by taking the average of 2005 to 2010 data.

This information is presented in Tables 4.3 and 4.4

Table 4.3 - Summary of Historical Annual Average Population and Loads

Year	Average Population[1]	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
2005	174275	11209.7	13104.0	2155.6	361.3
2006	174811	10303.1	12556.2	2410.3	367.1
2007	177404	11092.9	13736.7	2294.9	365.3
2008	180716	10704.9	12251.9	2379.3	405.5
2009	185139	12717.1	12933.9	2683.4	410.7
2010	188982	12882.7	13380.8	2655.5	368.2

[1] Based on data from Veolia PSR (2011), service population adjusted for seasonal redirection of flow from Windsor Park sewer district to NEWPCC from mid-Oct to mid-April. The population used for Windsor Park in the previous estimate was based on an estimate not actual census data. The current population estimate is based on actual census data and was found to be lower than that used in the previous assessment.

Table 4.4 - Summary of Calculated Historical Per Capita Loads (Annual Average Basis)

Year	Average Population[1]	TSS (Kg/Cap/day)	BOD_5 (Kg/Cap/day)	TKN (Kg/Cap/day)	TP (Kg/Cap/day)
2005	174,275	0.0643	0.0752	0.0124	0.0021
2006	174,811	0.0589	0.0718	0.0138	0.0021
2007	177,404	0.0625	0.0774	0.0129	0.0020
2008	180,716	0.0592	0.0678	0.0132	0.0022
2009	185,139	0.0687	0.0699	0.0145	0.0022
2010	188,982	0.0682	0.0708	0.0141	0.0019
Average (2005 ~ 2010)		0.0636	0.0722	0.0135	0.0021
Proposed Design Value		0.064	0.072	0.014	0.0021

[1] Based on data from Veolia PSR (2011), service population adjusted for seasonal redirection of flow from Windsor Park sewer district to NEWPCC from mid-Oct to mid-April.

Step 2 – Adjustment of per capita loads for Hauled Liquid Waste (HLW)

The HLW data provided by the City of Winnipeg Water and Waste Department was provided in two separate sets. One for constituent concentrations, and another for hauled volumes. An attempt was made to link the quality data with the volume data based on Load Ticket number to estimate the actual loads received by the SEWPCC. Unfortunately, due to limited cross-referencing it was not possible to directly link the data on a one-to-one basis. Nevertheless, it was possible to estimate average concentrations separately, which were considered to be representative of the typical HLW received at the SEWPCC for the period from June 2006 to December 2010, inclusive. However, the volume data was only complete for the years 2007 to 2009, inclusive. From this data it was possible to estimate the total annual volumes and partition the data into household (HH) and non-household (NHH) categories.

The average annual volume of HLW for the period 2007 to 2009, was found to be 50.8 ML per year or about 0.2 percent of the total annual flow to the SEWPCC (i.e., SEWPCC average flow: 63.3 ML/d) and can be ignored at this time for hydraulic design purposes). Correspondingly, the average population for the period 2007 to 2009 is about 184,000 people, accounting for Windsor Park seasonal operations. The following approach was used to account and adjust for the removal of HLW to the SEWPCC.

- The base load data includes the loads contributed through the existing HLW receiving facility.
- Stantec was informed by the Program Team that pending the HLW business case result, the practice of accepting HLW at the SEWPCC will be ended and that all HLW would be directed to the NEWPCC. As such, the SEWPCC expansion/upgrade is to be based on loads without the HLW component. Hence, the proportion of equivalent per capita constituent loading from HLW was removed from the base load.
- Based on historical HLW (June 2006 to December 2010) characteristics for the SEWPCC the total annual HLW load received at the SEWPCC were estimated. This annual HLW load was then divided by the average population of 184,000, to establish per capita contribution from HLW for each constituent parameter. These values were further adjusted by applying an 80% factor for partial truck loads from the trucks. This information is presented in Table 4.5.
- As demonstrated in Table 4.5, with the exception of TSS, the impact of HLW on the remaining parameters were insignificant. Due to the uncertainty of HLW sampling, it is recommended that only a 50% adjustment to the AA loading be applied and rounded up to two significant digits as conservativeness for the TSS loading.

Table 4.5 - Calculation of per capita loading from HLW

				BOD_5 (mg/L)	**TSS (mg/L)**	**TKN (mg/L N)**	**TP (mg/L P)**	
			HH	1,736	8,830	319	46	
			Non-HH	5,674	49,974	514	122	
	%	**Year total**		**BOD kg/year**	**TSS kg/year**	**TKN kg/year**	**TP kg/year**	
HH	53.4	27.1	ML	47,061	239,329	8,643	1,235	
NHH	46.6	23.7	ML	134,205	1,182,088	12,161	2,879	
	1.00	50.8	sum	181,266	1,421,418	20,804	4,113	kg/year
		population	184,000	0.0027	0.0212	0.0003	0.00006	kg/day
	Adjust for partial loads		80%	0.0022	**0.0169**	0.0002	0.0000	kg/day

Note: HH and NHH denotes House Hold waste and Non House Hold, respectively.

Based on the approach noted above, the projected per capita AA TSS loading was adjusted as follows: 0.064 – 50% of 0.0169 = 0.056, rounded to a value at 0.06 Kg/d/person. Based on a projected 2031 population of 270,000 people, the adjusted design per capita constituent loadings are presented in Table 4.6.

Table 4.6 - Projected AA Loadings (adjusted for HLW)

Design Year	Projected Population	TSS Load (Kg/d)	BOD loading (kg/d)	TKN loading (Kg/d)	TP Loading (Kg/d)
2031	270,000	16,200	19,440	3,780	567

Step 3 – Development of Seasonal Loadings

To be consistent with the Preliminary Design Report and Conceptual Design Report for the SEWPCC, the design year was broken into four (4) seasons as follows:

- Winter (December of previous year to February of the following)
- Spring (March to May)
- Summer (June to August)
- Fall (September to November)

Based on historical data for the years 2005 to 2010, the following constituent loadings were calculated for each season:

- Average loads
- Maximum month loads
- Maximum week loads
- Maximum day loads

The raw influent data is summarized in Tables 4.7, 4.8, 4.9 and 4.10. Following this, the data was normalized relative to AA loadings for the respective constituent to generate a series of load factors for the 2005 to 2010 period. This includes the following:

- Seasonal average to annual average (Table 4.11)
- Seasonal maximum month to annual average (Table 4.12)
- Seasonal maximum week to annual average (Table 4.13)
- Seasonal maximum day to annual average (Table 4.14)

A 98th percentile value was used to represent the design load factor from this range of values for each constituent. This information is presented in Table 4.15. Utilizing these seasonal load factors and the projected 2031 AA loads, the 2031 seasonal loadings for each constituent were developed. The data is summarized in Table 4.16 for the design year 2031. Based on the same seasonal factors and ignoring any changes to the per-capita constituent loading to the plant in future, Table 4.17 summarizes the projected plant loading for the year 2061 as a guideline for long-range planning purposes. An overall summary of current and projected, wastewater temperatures, flows, and loads are summarized in Table 4.18.

A review was conducted on the flow and quality data provided by the City of Winnipeg Water and Waste Department to quantify average concentrations for the constituents analyzed in the raw influent to the SEWPCC. To remove any unbalanced estimate of constituent concentrations associated with the simple averaging of measured concentrations, a flow-weighed averaging approach was used. On an annual average basis and for wet weather events, the flow weighted average would be more representative for load calculations. Since the vast majority of data collected is during dry weather conditions, the average concentrations would be more representative of normal dry weather flow to the SEWPCC, especially in the winter months. It is important to point out that in general, with the exception of nitrates, the flow weighted average tends to be lower than the simple averaging of constituents. As such it is important to properly characterize the constituent concentrations for process design purposes. For example, the VSS/TSS ratio is 82 percent based on average concentration, and 81.3 percent based on flow weighted average, indicating this ratio does not change significantly due to flow conditions.

The addition of solids processing at the SEWPCC will result a significant load to the main stream process. Reject water from the dewatering of digested sludge, referred to as centrate, has a small flow (i.e., ~ 1 to 2 percent of the influent flow) but can constitute 20 to 25 percent of the total nitrogen load, especially ammonia. Based on the effectiveness of centrate treatment as practiced at the NEWPCC, removal efficiency is expected to be on average greater than 95 percent for total nitrogen, including ammonia. The effective treatment of centrate will result in about 1percent (i.e., 5percent of 20percent = 1percent) additional nitrogen load and about 1 to 2 percent increase in the base flow on average. Since the flows and loads from treated centrate will be very small, they can be disregarded at this stage since they do not constitute a significant influence on the assessment of unit operations and treatment processes.

Table 4.7 – Historical Seasonal Average Loadings – Raw Influent

Season and Year	Avg Flow (ML)	TSS (Kg/d)	BOD_5 (Kg/d)	TKN (Kg/d)	NH3 (Kg/d)	TP (Kg/d)	ortho-P (Kg/d)
Spring							
2005	77.2	14896.6	14112.5	1565.3	ND	364.5	ND
2006	80.3	12804.2	12393.9	2643.7	1495.6	370.7	244.9
2007	68.1	12331.9	14300.4	2439.1	1595.1	391.0	242.7
2008	55.7	10551.0	12089.0	2539.5	ND	436.3	263.2
2009	81.2	16263.3	13156.5	3108.4	ND	452.6	257.4
2010	70.6	14032.4	14545.0	2691.8	ND	411.5	236.7
2011	87.7	15547.0	13878.1	2530.8	1570.1	379.4	218.3
Summer							
2005	91.6	11563.5	12577.7	2122.2	ND	349.7	ND
2006	55.0	9873.8	11916.9	2269.8	1361.8	363.9	209.7
2007	65.9	11150.4	13536.3	2257.4	ND	366.1	221.3
2008	66.9	12374.6	12606.2	2362.4	ND	402.2	261.9
2009	74.1	13275.2	13276.0	2680.6	ND	402.0	253.0
2010	76.9	13520.3	13163.3	2550.1	ND	316.4	203.2
Fall							
2005	56.2	11479.4	13384.3	2248.0	ND	372.6	ND
2006	52.6	9314.7	12864.7	2372.1	1566.4	368.7	233.5
2007	52.9	10390.7	13407.3	2133.3	ND	334.1	245.8
2008	62.3	10302.4	12487.4	2362.7	ND	411.7	283.5
2009	55.3	11072.6	13023.2	2681.7	ND	401.7	248.0
2010	73.2	12508.8	12984.9	2779.7	ND	381.7	222.9
Winter							
2006	49.7	9824.1	13404.2	2341.8	ND	356.2	ND
2007	47.1	10026.2	13994.7	2390.9	1510.2	382.0	231.7
2008	47.2	9935.9	11857.7	2257.0	ND	354.3	242.0
2009	48.6	9639.6	12009.6	2379.0	ND	403.4	255.5
2010	45.9	10788.1	13163.5	2574.1	ND	376.8	229.7
2011	47.9	11649.5	12675.4	2605.6	1536.2	363.9	222.5

ND = no data available

Table 4.8 – Historical Seasonal Maximum Month Loadings – Raw Influent

	Flow (ML/d)			TSS (Kg/d)			BOD (Kg/d)			TKN (Kg/d -N)			Ammonia (Kg/d -N)			TP (Kg/d -P)		
Sean and Year	Avg	Max 30d	Min 30d	Avg	Max 30d	Min 30d	Avg	Max 30d	Min 30d	Avg	Max 30d	Min 30d	Avg	Max 30d	Min 30d	Avg	Max 30d	Min 30d
Spring																		
2005	77.2	103.6	58.0	14896.6	ND	ND	14112.5	ND	ND	2347.9		ND	ND	ND	ND	364.5	ND	ND
2006	80.3	122.3	62.1	12804.2	15916.8	11364.7	12393.9	13482.7	10893.5	2643.7	2955.3	2459.9	1495.6	1603.6	1446.7	370.7	417.2	341.5
2007	68.1	81.4	58.0	12331.9	14260.9	11337.1	14300.4	14835.7	13848.6	2439.1	2538.3	2367.0	1595.1	1658.6	1471.0	391.0	423.7	377.5
2008	55.7	60.5	50.0	10551.0	11613.0	9725.4	12089.0	12995.8	11259.3	2539.5	2640.9	2454.8	ND	ND	ND	436.3	471.8	417.5
2009	81.2	103.0	63.0	16263.3	17382.4	14676.0	13156.5	14596.6	10374.2	3108.4	3256.2	2983.8	ND	ND	ND	452.6	479.1	433.9
2010	70.6	88.2	60.6	14032.4	15256.0	12322.5	14545.0	15715.8	13511.0	2691.8	2935.5	2413.8	ND	ND	ND	411.5	419.2	392.3
2011	87.7	114.4	57.0	15547.0	17586.9	13772.3	13878.1	15117.0	13356.6	2530.8	2648.7	2354.6	1570.1	1610.9	1514.9	379.4	394.0	366.1
Summer																		
2005	91.6	124.4	60.9	11563.5	12367.5	9419.7	12577.7	13395.2	11303.1	2122.2	2233.1	1964.0	ND	ND	ND	349.7	354.0	338.2
2006	55.0	57.9	50.4	9873.8	11211.7	8587.8	11916.9	13068.9	11072.8	2269.8	2363.1	2101.9	1361.8	1501.8	1232.7	363.9	383.6	331.3
2007	65.9	84.5	51.1	11150.4	12901.7	9678.4	13536.3	14384.9	12721.6	2257.4	2360.0	2066.2	ND	ND	ND	366.1	382.7	324.3
2008	66.9	81.6	52.4	12374.6	14984.2	9956.6	12606.2	13793.8	11500.6	2362.4	2396.3	2256.7	ND	ND	ND	402.2	431.9	367.0
2009	74.1	83.1	68.4	13275.2	15139.5	11461.2	13276.0	15064.4	11705.8	2680.6	2827.8	2418.9	ND	ND	ND	402.0	433.6	362.7
2010	76.9	84.9	71.8	13520.3	14583.2	12232.4	13163.3	13646.3	12626.6	2550.1	2700.1	2435.3	ND	ND	ND	316.4	395.3	188.9
Fall																		
2005	56.2	59.3	52.5	11479.4	12021.9	9841.7	13384.3	13959.9	12496.4	2248.0	2364.0	1963.1	ND	ND	ND	372.6	378.1	353.5
2006	52.6	55.2	50.6	9314.7	9887.8	8949.4	12864.7	13386.5	12596.6	2372.1	2434.4	2288.9	1566.4	1642.3	1559.2	368.7	378.3	358.3
2007	52.9	58.3	49.0	10390.7	11153.4	9619.5	13407.3	14044.2	12541.1	2133.3	2283.5	1852.7	ND	ND	ND	334.1	352.0	307.2
2008	62.3	66.1	59.8	10302.4	10957.6	9667.4	12487.4	12900.7	12003.8	2362.7	2460.2	2263.6	ND	ND	ND	411.7	429.8	397.5
2009	55.3	61.1	50.3	11072.6	12208.4	10312.2	13023.2	13944.5	12745.8	2681.7	2895.3	2640.5	ND	ND	ND	401.7	422.8	395.3
2010	73.2	87.4	60.0	12508.8	13292.3	11392.7	12984.9	13424.4	12524.0	2779.7	2897.5	2681.3	ND	ND	ND	381.7	397.9	367.6
Winter																		
2005	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
2006	49.7	50.8	48.6	9824.1	10414.4	9252.7	13404.2	13806.2	12783.8	2341.8	2372.3	2275.6	ND	ND	ND	356.2	367.2	334.3
2007	47.1	48.5	45.7	10026.2	12695.4	8377.2	13994.7	15777.0	13220.6	2390.9	2633.2	2248.4	1510.2	1584.7	1394.6	382.0	413.0	361.3
2008	47.2	47.9	46.7	9935.9	10505.8	8813.7	11857.7	12577.6	10701.3	2257.0	2401.9	2059.7	ND	ND	ND	354.3	364.9	332.9
2009	48.6	49.7	47.2	9639.6	10837.2	8475.7	12009.6	12482.3	11611.1	2379.0	2440.1	2332.1	ND	ND	ND	403.4	428.1	378.9
2010	45.9	46.9	44.8	10788.1	11535.4	10095.0	13163.5	13596.8	12762.5	2574.1	2630.9	2479.9	ND	ND	ND	376.8	386.4	355.7
2011	47.9	49.3	46.4	11649.5	11948.7	11059.9	12675.4	13262.3	12151.5	2605.6	2678.7	2556.4	1536.2	1548.7	1534.8	363.9	378.9	343.4

ND = No data available

Note: Data provided by City of Winnipeg for period Jan 1, 2005 to June 30, 2011 inclusive

Table 4.9 - Historical Seasonal Maximum Week Loadings – Raw Influent

	Flow (ML/d)			TSS (Kg/d)			BOD (Kg/d)			TKN (Kg/d -N)			Ammonia (Kg/d -N)			TP (Kg/d -P)		
Season and Year	Avg	Max 7d	Min 7d	Avg	Max 7d	Min 7d	Avg	Max 7d	Min 7d	Avg	Max 7d	Min 7d	Avg	Max 7d	Min 7d	Avg	Max 7d	Min 7d
Spring																		
2005	77.2	137.1	50.4	14896.6	ND	ND	14112.5	ND	ND	2347.9	ND	ND	ND	ND	ND	364.5	ND	ND
2006	80.3	122.3	62.1	12804.2	15916.8	11364.7	12393.9	13482.7	10893.5	2643.7	2955.3	2459.9	1495.6	1603.6	1446.7	370.7	417.2	341.5
2007	68.1	118.9	46.5	12331.9	20167.8	9499.7	14300.4	16958.9	12774.7	2439.1	2651.8	2242.3	1595.1	1728.2	1161.7	391.0	447.2	335.8
2008	55.7	64.4	46.8	10551.0	13437.7	7806.2	12089.0	14018.3	10540.0	2539.5	3365.1	2223.0	ND	ND	ND	436.3	682.0	372.8
2009	81.2	153.3	47.8	16263.3	25781.8	11289.6	13156.5	21742.0	8349.7	3108.4	3781.0	2716.2	ND	ND	ND	452.6	616.2	396.8
2010	70.6	144.3	48.1	14032.4	23799.0	10966.3	14545.0	20293.8	12543.7	2691.8	3111.0	2162.4	ND	ND	ND	411.5	476.4	366.7
2011	87.7	159.4	47.3	15547.0	21678.6	12514.9	13878.1	17969.6	11597.2	2530.8	2738.9	2155.5	1570.1	1662.7	1398.5	379.4	435.2	350.1
Summer																		
2005	91.6	168.0	58.1	11563.5	17601.9	7794.8	12577.7	19813.7	7914.9	2122.2	2357.6	1485.7	ND	ND	ND	349.7	400.6	285.6
2006	55.0	70.5	49.4	9873.8	16920.0	7579.8	11916.9	13570.9	10266.7	2269.8	2571.6	1986.8	1361.8	1604.2	1185.6	363.9	474.5	302.3
2007	65.9	102.1	49.5	11150.4	17228.3	8672.0	13536.3	16669.2	11850.5	2257.4	2963.1	1983.1	ND	ND	ND	366.1	481.7	303.8
2008	66.9	121.9	43.8	12374.6	24782.4	8470.9	12606.2	16475.3	10289.4	2362.4	2735.0	1950.3	ND	ND	ND	402.2	492.4	327.0
2009	74.1	103.0	58.0	13275.2	18862.7	8894.7	13276.0	17298.6	9813.0	2680.6	3063.0	2385.6	ND	ND	ND	402.0	467.4	310.3
2010	76.9	99.4	58.5	13520.3	18196.4	10938.9	13163.3	14711.4	11488.9	2550.1	2916.0	2296.2	ND	ND	ND	316.4	410.0	172.0
Fall																		
2005	56.2	64.8	50.5	11479.4	14554.4	8355.4	13384.3	15540.1	11063.7	2248.0	2996.1	1763.8	ND	ND	ND	372.6	418.1	326.2
2006	52.6	61.9	48.1	9314.7	11125.3	7227.2	12864.7	14143.5	11309.5	2372.1	2675.5	2128.0	1566.4	1744.0	1279.6	368.7	428.1	333.2
2007	52.9	64.8	48.1	10390.7	13448.3	8309.7	13407.3	14903.1	11755.7	2133.3	2761.5	1729.4	ND	ND	ND	334.1	388.2	272.1
2008	62.3	77.7	53.7	10302.4	12604.1	8374.9	12487.4	14094.4	10351.9	2362.7	2651.8	2117.1	ND	ND	ND	411.7	477.8	369.4
2009	55.3	72.4	46.6	11072.6	13717.5	9026.7	13023.2	15431.5	11233.5	2681.7	3349.8	2233.1	ND	ND	ND	401.7	514.1	357.6
2010	73.2	112.7	51.8	12508.8	16106.7	10389.9	12984.9	14737.7	11472.8	2779.7	3122.8	2210.7	ND	ND	ND	381.7	419.0	336.6
Winter																		
2005	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
2006	49.7	51.8	47.0	9824.1	11486.2	7751.6	13404.2	15169.1	11718.2	2341.8	2470.5	2178.1	ND	ND	ND	356.2	401.8	297.7
2007	47.1	52.2	44.2	10026.2	21099.5	7059.0	13994.7	20578.2	11773.4	2390.9	2932.8	2107.9	1510.2	1707.1	1253.5	382.0	493.2	343.4
2008	47.2	48.8	45.2	9935.9	11457.3	7435.6	11857.7	13269.9	9915.0	2257.0	2878.7	1935.8	ND	ND	ND	354.3	400.0	290.9
2009	48.6	55.9	45.9	9639.6	12225.1	7637.6	12009.6	13393.8	10557.9	2379.0	2530.7	2172.8	ND	ND	ND	403.4	488.3	350.7
2010	45.9	47.9	43.8	10788.1	12852.7	8602.1	13163.5	14824.7	11763.9	2574.1	2772.9	2368.9	ND	ND	ND	376.8	405.5	343.0
2011	47.9	51.4	45.7	11649.5	13661.1	9714.9	12675.4	13726.7	11194.2	2605.6	2761.0	2417.7	1536.2	1598.8	1458.2	363.9	392.8	315.8

ND = No data available
Note: Data provided by City of Winnipeg for period Jan 1, 2005 to June 30, 2011 inclusive

Table 4.10 – Historical Seasonal Maximum Day Loadings – Raw Influent

	Year	TSS (Kg/d)	BOD (Kg/d)	TKN (Kg/d)	NH3 (Kg/d)	TP (Kg/d)
Spring						
	2005	23957.0	19764.5	2695.2	ND	407.3
	2006	50130.0	18453.6	4647.5	1778.4	788.2
	2007	29185.6	20569.2	3354.8	1802.9	598.6
	2008	20563.2	16234.2	4444.0	ND	878.7
	2009	35850.0	33196.0	4725.0	ND	808.4
	2010	32510.5	32144.3	3638.0	ND	751.3
	2011	36496.2	25294.4	2973.4	1903.8	502.8
Summer						
	2005	27783.8	32832.2	2689.6	ND	478.8
	2006	43928.0	23617.4	3074.0	1604.2	995.5
	2007	27899.0	27634.1	4576.0	ND	755.0
	2008	44940.0	29357.4	3226.2	ND	731.6
	2009	32782.6	30022.0	4399.8	ND	785.1
	2010	45138.5	23816.0	3761.5	ND	540.6
Fall						
	2005	24618.4	21536.0	3107.5	ND	502.2
	2006	17472.0	18508.8	3686.4	1744.0	603.6
	2007	27472.0	21918.4	3225.6	ND	602.0
	2008	19423.6	18432.6	2868.1	ND	743.3
	2009	20386.7	19035.5	4125.4	ND	590.1
	2010	22965.9	20505.3	3691.0	ND	474.9
Winter						
	2005	ND	ND	ND	ND	ND
	2006	16789.5	19473.5	2671.2	ND	422.8
	2007	26692.8	22758.6	3074.5	1728.0	537.6
	2008	18808.8	16381.5	4037.5	ND	479.2
	2009	18837.4	18232.6	3010.1	ND	784.8
	2010	18043.9	16774.4	2996.5	ND	456.2
	2011	18234.8	15827.5	2921.6	1683.1	484.9

ND = No data available

Note: Data provided by City of Winnipeg for period Jan 1, 2005 to June 30, 2011 inclusive

Table 4.11 – Load Factors – Ratio of Seasonal Average to Annual Average

Season	and Year	TSS Loading Factor	BOD Loading Factor	TKN Loading Factor	TP Loading Factor
Spring	2005	1.329	1.077	0.726	1.009
	2006	1.243	0.987	1.097	1.010
	2007	1.112	1.041	1.063	1.070
	2008	0.986	0.987	1.067	1.076
	2009	1.279	1.017	1.158	1.102
	2010	1.089	1.087	1.014	1.118
	98% tile	**1.324**	**1.086**	**1.152**	**1.116**
Summer	2005	1.032	0.960	0.984	0.953
	2006	0.958	0.949	0.942	0.996
	2007	1.005	0.985	0.984	0.903
	2008	1.156	1.029	0.993	0.979
	2009	1.044	1.026	0.999	1.092
	2010	1.049	0.984	0.960	0.859
	98% tile	**1.145**	**1.029**	**0.998**	**1.082**
Fall	2005	1.024	1.021	1.043	1.031
	2006	0.904	1.025	0.984	1.004
	2007	0.937	0.976	0.930	0.915
	2008	0.962	1.019	0.993	1.015
	2009	0.871	1.007	0.999	0.978
	2010	0.971	0.970	1.047	1.037
	98% tile	**1.019**	**1.024**	**1.046**	**1.036**
Winter	2006	0.954	1.068	0.972	0.970
	2007	0.904	1.019	1.042	1.046
	2008	0.928	0.968	0.949	0.874
	2009	0.758	0.929	0.887	0.982
	2010	0.837	0.984	0.969	1.023
	98% tile	**0.951**	**1.064**	**1.036**	**1.044**

Table 4.12 – Load Factors – Ratio of Seasonal Max 30d to Annual Average

Season and Year		TSS Loading Factor	BOD Loading Factor	TKN Loading Factor	TP Loading Factor
Spring	2006	1.545	1.074	1.226	1.137
	2007	1.286	1.080	1.106	1.160
	2008	1.085	1.061	1.110	1.163
	2009	1.367	1.129	1.213	1.167
	2010	1.184	1.174	1.105	1.138
	98% tile	1.531	1.171	1.225	1.166
Summer	2005	1.103	1.022	1.036	0.980
	2006	1.088	1.041	0.980	1.045
	2007	1.163	1.047	1.028	1.047
	2008	1.400	1.126	1.007	1.065
	2009	1.190	1.165	1.054	1.056
	2010	1.132	1.020	1.017	1.074
	98% tile	1.379	1.161	1.052	1.073
Fall	2005	1.072	1.065	1.097	1.047
	2006	0.960	1.066	1.010	1.031
	2007	1.005	1.022	0.995	0.963
	2008	1.024	1.053	1.034	1.060
	2009	0.960	1.078	1.079	1.029
	2010	1.032	1.003	1.091	1.081
	98% tile	1.068	1.077	1.096	1.079
Winter	2006	1.011	1.100	0.984	1.000
	2007	1.144	1.149	1.147	1.131
	2008	0.981	1.027	1.009	0.900
	2009	0.852	0.965	0.909	1.042
	2010	0.895	1.016	0.991	1.050
	98% tile	1.134	1.145	1.136	1.124

Table 4.13 – Load Factors – Ratio of Seasonal Max 7d to Annual Average

Season	and Year	TSS Loading Factor	BOD Loading Factor	TKN Loading Factor	TP Loading Factor
Spring	2006	1.545	1.074	1.226	1.137
	2007	1.818	1.235	1.156	1.224
	2008	1.255	1.144	1.414	1.682
	2009	2.027	1.681	1.409	1.500
	2010	1.847	1.517	1.172	1.294
	98% tile	2.013	1.668	1.414	1.667
Summer	2005	1.570	1.512	1.094	1.109
	2006	1.642	1.081	1.067	1.293
	2007	1.553	1.213	1.291	1.319
	2008	2.315	1.345	1.150	1.214
	2009	1.483	1.337	1.141	1.138
	2010	1.412	1.099	1.098	1.114
	98% tile	2.248	1.495	1.277	1.316
Fall	2005	1.298	1.186	1.390	1.157
	2006	1.080	1.126	1.110	1.166
	2007	1.212	1.085	1.203	1.063
	2008	1.177	1.150	1.115	1.178
	2009	1.079	1.193	1.248	1.252
	2010	1.250	1.101	1.176	1.138
	98% tile	1.294	1.192	1.376	1.245
Winter	2006	1.115	1.208	1.025	1.095
	2007	1.902	1.498	1.278	1.350
	2008	1.070	1.083	1.210	0.986
	2009	0.961	1.036	0.943	1.189
	2010	0.998	1.108	1.044	1.101
	98% tile	1.839	1.475	1.273	1.337

Table 4.14 – Load Factors – Ratio of Seasonal Max Day to Annual Average

	Season and Year	TSS Loading Factor	BOD Loading Factor	TKN Loading Factor	TP Loading Factor
Spring	2005	2.137	1.508	1.250	1.127
	2006	4.866	1.470	1.928	2.147
	2007	2.631	1.497	1.462	1.639
	2008	1.921	1.325	1.868	2.167
	2009	2.819	2.567	1.761	1.968
	2010	2.524	2.402	1.370	2.041
	98% tile	4.661	2.550	1.922	2.165
Summer	2005	2.479	2.506	1.248	1.304
	2006	4.264	1.881	1.275	2.725
	2007	2.515	2.012	1.994	1.862
	2008	4.198	2.396	1.356	1.781
	2009	2.578	2.321	1.640	2.132
	2010	3.504	1.780	1.416	1.468
	98% tile	4.257	2.495	1.959	2.666
Fall	2005	2.196	1.643	1.442	1.390
	2006	1.696	1.474	1.529	1.644
	2007	2.477	1.596	1.406	1.648
	2008	1.814	1.504	1.205	1.833
	2009	1.603	1.472	1.537	1.437
	2010	1.783	1.532	1.390	1.290
	98% tile	2.448	1.639	1.537	1.814
Winter	2006	1.630	1.551	1.108	1.152
	2007	2.406	1.657	1.340	1.472
	2008	1.757	1.337	1.697	1.182
	2009	1.481	1.410	1.122	1.911
	2010	1.401	1.254	1.128	1.239
	98% tile	2.354	1.648	1.668	1.876

Table 4.15 – Summary of Seasonal Load Factors Relative to Annual Average Loadings

Season	Averaging Period	TSS	BOD5	TKN	TP
Spring	Average	1.324	1.086	1.152	1.116
	Max 30d rolling avg	1.531	1.171	1.225	1.166
	Max 7d rolling avg	2.013	1.668	1.414	1.667
	Max Day	4.661	2.550	1.922	2.165
Summer	Average	1.145	1.029	0.998	1.082
	Max 30d rolling avg	1.379	1.161	1.052	1.073
	Max 7d rolling avg	2.248	1.495	1.277	1.316
	Max Day	4.257	2.495	1.959	2.666
Fall	Average	1.019	1.024	1.046	1.036
	Max 30d rolling avg	1.068	1.077	1.096	1.079
	Max 7d rolling avg	1.294	1.192	1.376	1.245
	Max Day	2.448	1.639	1.537	1.814
Winter	Average	0.951	1.064	1.044	1.044
	Max 30d rolling avg	1.134	1.145	1.136	1.124
	Max 7d rolling avg	1.839	1.475	1.273	1.337
	Max Day	2.354	1.648	1.668	1.876
Annual Average		1.000	1.000	1.000	1.000

Table 4.16 – Summary of Projected 2031 Seasonal Loadings (based on a population of 270,000)

Season	Average Period	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	TKN Loading (Kg/d-N)	TP Loading (Kg/d-P)
Spring	Average	21447	21112	4355	633
	Max 30d rolling avg	24796	22761	4631	661
	Max 7d rolling avg	32610	32423	5345	945
	Max Day	75506	49575	7266	1227
Summer	Average	18554	19997	3774	614
	Max 30d rolling avg	22337	22567	3977	608
	Max 7d rolling avg	36414	29069	4827	746
	Max Day	68964	48494	7403	1511
Fall	Average	16504	19911	3955	588
	Max 30d rolling avg	17308	20936	4143	612
	Max 7d rolling avg	20956	23180	5201	706
	Max Day	39666	31856	5808	1029
Winter	Average	15414	20677	3946	592
	Max 30d rolling avg	18367	22251	4296	637
	Max 7d rolling avg	29793	28671	4810	758
	Max Day	38141	32043	6306	1064
Annual Average		16200.0	19440.0	3780.0	567.0

Table 4.17 – Summary of Projected 2031 Seasonal Loadings (based on a population of 400,000)

Season	Average Period	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	TKN Loading (Kg/d-N)	TP Loading (Kg/d-P)
Spring	Average	31773	31277	6452	937
	Max 30d rolling avg	36735	33720	6861	980
	Max 7d rolling avg	48311	48034	7918	1400
	Max Day	111861	73444	10764	1818
Summer	Average	27488	29626	5591	909
	Max 30d rolling avg	33092	33432	5891	901
	Max 7d rolling avg	53946	43065	7151	1106
	Max Day	102168	71843	10968	2239
Fall	Average	24450	29498	5860	870
	Max 30d rolling avg	25641	31016	6138	906
	Max 7d rolling avg	31045	34341	7704	1045
	Max Day	58764	47194	8605	1524
Winter	Average	22836	30633	5846	877
	Max 30d rolling avg	27210	32965	6364	944
	Max 7d rolling avg	44138	42476	7126	1123
	Max Day	56504	47471	9343	1576
	Annual Average	24000.0	28800.0	5600.0	840.0

Table 4.18 **Summary of Current and Projected Wastewater Temperatures, Flows, and Loads**

	Last 6 year (2005-10)					Projected 2031*					Projected 2061*				
	Winter	Spring	Summer	Fall	Annual Ave	Winter	Spring	Summer	Fall	Annual Ave	Winter	Spring	Summer	Fall	Annual Ave
TSS (kg/d)					11,477					16,200					24,000
Ave Day	10,958	15,153	13,191	11,733		15,414	21,447	18,554	16,504		22,836	31,773	27,488	24,450	
Max 30 day rolling ave	13,058	17,567	15,880	12,305		18,367	24,796	22,337	17,308		27,210	36,735	33,092	25,641	
Max 7 day rolling ave	21,181	21,190	25,887	14,898		29,793	32,610	36,414	20,956		44,138	48,311	53,946	31,045	
Max Day	27,115	53,494	49,028	28,199		38,141	75,506	68,964	39,666		56,504	111,861	102,168	58,764	
TKN (Kg/d)					2,469					3,780					5,600
Ave Day	2,577	2,844	2,464	2,583		3,946	4,355	3,774	3,955		5,846	6,452	5,591	5,860	
Max 30 day rolling ave	2,805	3,024	2,597	2,706		4,296	4,631	3,977	4,143		6,364	6,861	5,891	6,138	
Max 7 day rolling ave	3,141	3,490	3,152	3,396		4,810	5,345	4,827	5,201		7,126	7,918	7,151	7,704	
Max Day	4,118	4,745	4,835	3,793		6,306	7,266	7,403	5,808		9,343	10,764	10,968	8,605	
TP (Kg/d)					383					567					840
Ave Day	399	427	414	396		592	633	614	588		877	937	909	870	
Max 30 day rolling ave	430	446	410	413		637	661	608	612		944	980	901	906	
Max 7 day rolling ave	512	638	504	476		758	945	746	706		1,123	1,400	1,106	1,045	
Max Day	718	828	1,020	694		1,064	1,227	1,511	1,029		1,576	1,818	2,239	1,524	
BOD (Kg/d)					12,994					19,440					28,800
Ave Day	13,821	14,112	13,367	13,309		20,677	21,112	19,997	19,911		30,633	31,277	29,626	29,498	
Max 30 day rolling ave	14,873	15,214	15,084	13,994		22,251	22,761	22,567	20,936		32,965	33,720	33,432	31,016	
Max 7 day rolling ave	19,165	21,673	19,430	15,494		28,671	32,423	29,069	23,180		42,476	48,034	43,065	34,341	
Max Day	21,419	33,137	32,415	21,293		32,043	49,575	48,494	31,856		47,471	73,444	71,843	47,194	
Temperature					15					15.1					15.1
Min Day	8.7	8.2	12.9	13.0		8.7	8.2	12.9	13.0		8.7	8.2	12.9	13.0	
Min 7 day rolling ave	9.8	8.7	13.1	15.1		9.8	8.7	13.1	15.1		9.8	8.7	13.1	15.1	
Min 30 day rolling ave	11.2	9.8	13.1	15.0		11.2	9.8	13.1	15.0		11.2	9.8	13.1	15.0	
Average	13.9	12.8	16.5	17.0		13.9	12.8	16.5	17.0		13.9	12.8	16.5	17.0	
Flow (ML/d)					63					95					136
Min Hour	20	20	20	20		35	35	35	35		58	58	58	58	
Min Day	40	45	33	45		68	73	60	73		110	115	101	115	
Average	48	71	72	59		75	108	105	89		112	153	149	128	
Max Hour	174	355	350	302		236	420	420	335		334	718	686	446	
Max Day	79	276	272	205		114	300	324	272		161	513	529	363	
Max 7 day rolling ave	56	159	168	114		85	201	220	155		125	283	315	212	
Max 30 day rolling ave	51	112	114	91		80	160	174	124		117	219	238	173	

** denotes: Removed TSS load associated with Hauled Liquid Waste*

Table 4.19 provides a summary of the concentrations for specific parameters in the raw influent from 2005 to 2010, inclusive.

Table 4.19 – Average Concentrations for Specific Parameters

Raw Influent Parameter	Average Concentration (mg/L)	Flow Weighted Average Concentration (mg/L)
Total Suspended Solids (TSS)	193.9	184.9
Total Kjeldahl Nitrogen (TKN)	42.5	39.2
Ammonia (NH4+NH3)	25.7	24.3
Nitrite + Nitrate (NO2+NO3)	0.6	0.9
Total Nitrogen (TN)	41.3	41.1
Total Phosphorus (TP)	6.6	6.0
Soluble Phosphorus (Ortho-P)	4.1	3.8
Particulate (Part-P)	2.5	2.4
Carbonaceous BOD5 (CBOD5)	150.1	144.7
5-day Biochemical Oxygen Demand (BOD5)	223.8	205.3
Total Organic Carbon (TOC)	151.4	139.9
Chemical Oxygen Demand (COD)	501.5	437.7
Volatile Suspended Solids (VSS)	159.1	150.4
Soluble Organic Carbon (SOC)	65.0	59.4
Volatile Fatty Acids (VFA)	25.2	21.3

5.0 Biosolids Handling and Treatment

5.1 BACKGROUND

The inclusion of onsite solids processing at the SEWPCC is an important consideration in the design of the solids handling and liquid train treatment process. As part of the SEWPCC Project Definition work plan we have assumed that anaerobic digestion and dewatering will be located at the SEWPCC and included in the overall plant design. For this scenario, the Program Team has also decided that biosolids from the West End WPCC (WEWPCC) will continue to be processed at the NEWPCC.

A preliminary mass balance analysis was performed on the plant processes to assess the impact of including solids treatment at the SEWPCC, particularly on the effect of return streams such as centrate loading to the liquid treatment train. The mass balance was explicitly done to develop preliminary sizing of the digesters and associated solids handling facility. Based on the analysis of flows and loads, the 2031 maximum month spring conditions represents the greatest projected treatment condition that needs to be considered in the design of the solids handling facility and was accordingly selected for this analysis.

It is our understanding that the Program Team is undertaking a Biosolids Master Plan for the City's three wastewater treatment facilities. The selected biosolids implementation strategy if different from our assumptions stated above may have an impact on SEWPCC design and should be revisited prior to the preliminary design. This would reduce the risks associated with advancing the SEWPCC upgrades in subsequent engineering design phases.

The following sections provide a summary of the assumptions for developing the mass balance around each of the unit processes. Figure 5.1 provides an overall summary of the mass balance for the unit processes evaluated. Additional calculations are shown in Tables A to M provided in Appendix E

5.2 PROCESS DESCRIPTION

The analysis of the mass balance considers the flows and loadings associated with the maximum-month conditions during spring. The corresponding flows and loadings from Table 4.18 are as follows:

- Flow = 160 MLD
- TSS = 24,796 kg/d
- BOD5 = 22,761 kg/d
- TKN = 4,631 kg/d
- TP = 661 kg/d

5.2.1 Raw Wastewater Pumping and Headworks

For the purpose of this mass balance assessment, no removal of any constituents was assumed that occurred through the pumping and the headworks. Headworks consist of 6 mm perforated screening followed by grit removal. Based on data obtained from the City, the volatile suspended solids (VSS) present in the raw wastewater stream was estimated at 81.3% of TSS and the ammonia fraction of the TKN estimated at 62%. For the purpose of this mass balance, treated centrate is returned to the headworks and is blended with the incoming flow. The resulting flows and loads are shown in Figure 5.1 prior to the splitter box.

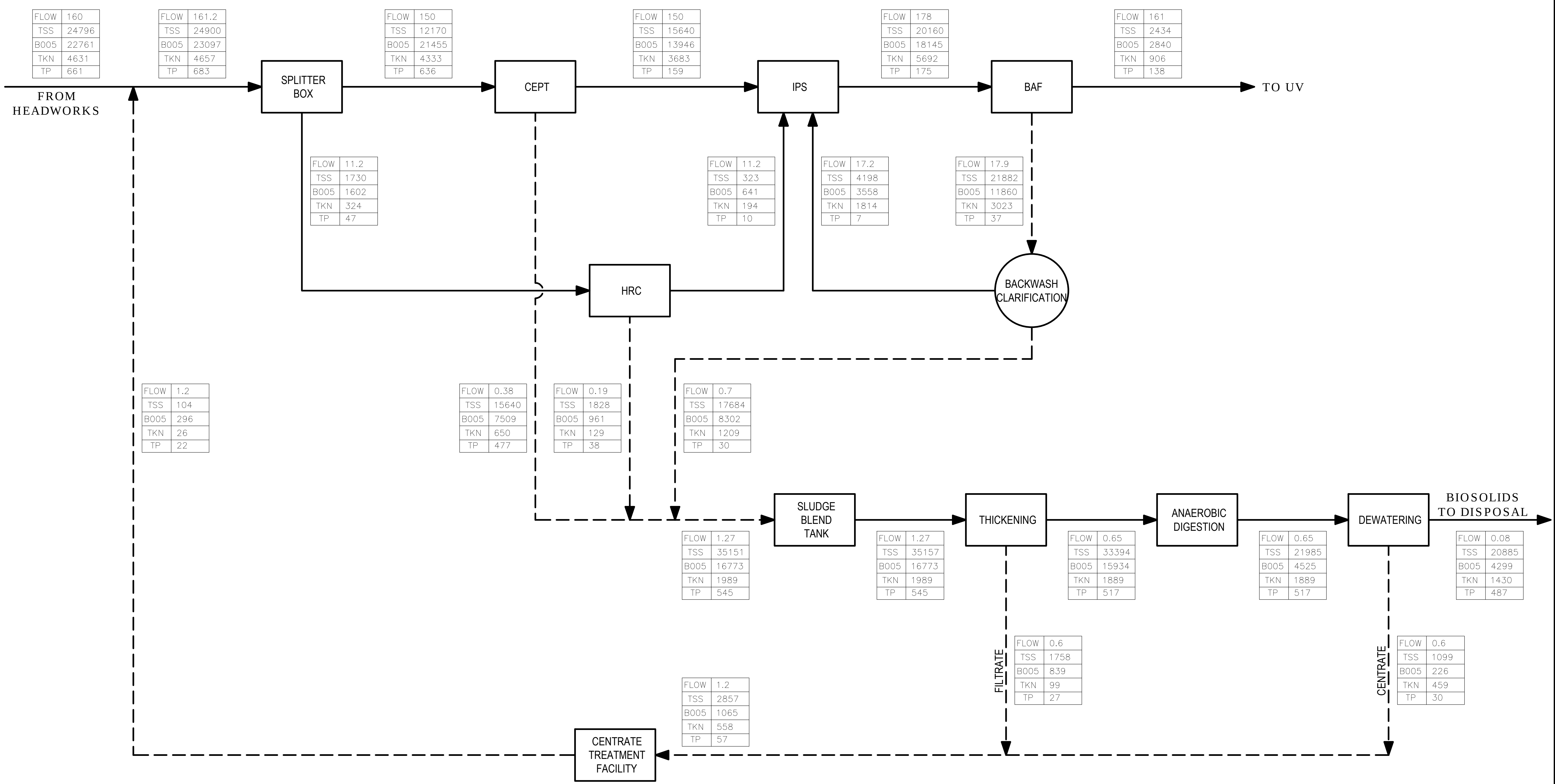
5.2.2 High Rate Clarification (HRC)

Following headworks, the wastewater stream is split into two flow paths. Flows up to 150 ML/d will be directed to the chemically enhanced primary treatment (CEPT) process while flows greater than 150 ML/d but less than equal to 325 ML/d will be directed to a sidestream high-rate clarification (HRC) process.

For maximum month spring flows with centrate stream returning to the headworks, HRC will receive approximately 11.2 ML/d. Refer to Figure 5.1 for associated loads. To minimize the solids loading on the BAF process, waste activated sludge (WAS) from centrate treatment can either be sent to the CEPT treatment process, the HRC treatment process, or to the sludge thickening and blend tank. Since the centrate treatment process is yet to be selected and designed, it is not possible to estimate the amount and type of WAS that will be generated. Due to the range of possible options associate with centrate treatment, additional details need to be developed during the schematic design that specifically addresses this feedback loop from the digestion and dewatering of sludge. For completeness and illustrative purposes only, the WAS from the centrate treatment process is shown as going to the sludge thickening and blend tank. It should be noted that this flow splitting strategy is based on the proposed maximum CEPT capacity of 150 ML/d. This capacity will be revisited in the future following stress testing during schematic/preliminary design. The removal efficiencies associated with the HRC process are summarized as follows:

- TSS = 85%
- BOD5 = 60%
- TKN = 40%
- TP = 80%

The HRC process will utilize a chemical coagulant such as ferric chloride and a coagulant aid such as a polymer. Ferric chloride will produce chemical sludge as it reacts with suspended particles in the wastewater increasing the total sludge production which is reflected in the calculations. Settled sludge is diverted to the sludge blend tank. The following chemical feed and sludge concentrations were assumed for the mass balance.



NOTE:

- ALL PLANT FLOWS (Q) IS IN ML/d
- ALL CONSISTUENT LOADINGS (TSS, COD, TKN, AND TP) ARE IN Kg/d
- BAF - EFFLUENT AMMONIA IS 4 mg/L
- CEPT IS CHEMICALLY ENHANCED PRIMARY TREATMENT
- IPS IS INTERMEDIATE PUMP STATION
- HRC IS HIGH RATE CLARIFICATION
- BAF IS BIOLOGICALLY ACTIVE FILTRATION

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO. REVISIONS DATE BY

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 Fax 204-453-9012

DESIGNED BY | CHECKED BY
DRAWN BY | APPROVED BY
HOR. SCALE: VERTICAL: | RELEASED FOR CONSTRUCTION:
DATE 2012.03.05 | DATE

ENGINEER'S SEAL

TENDER NO. 2007-

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

MASS BALANCE DIAGRAM
MAXIMUM MONTH FLOW - SPRING

CITY DRAWING NUMBER FIG. 5.1

SHEET OF

01/000

- Polymer feed = 1 mg/L
- Ferric chloride = 75 mg/L
- HRC Sludge Conc. = 10 g/L

Table A in Appendix E presents conceptual design data for the HRC process.

5.2.3 Chemically Enhanced Primary Treatment (CEPT)

As stated earlier, plant flows up to a maximum of 150 ML/d are treated the CEPT process. The CEPT process is similar to the HRC process, except that it operates with a much lower surface overflow rate. The following removal efficiencies for TSS, BOD5, TKN and TP as discussed in Table 6.11 of this report was utilized for the mass balance.

- TSS = 50%
- BOD5 = 35%
- TKN = 15%
- TP = 75%

Similar to HRC, ferric chloride and a polymer is utilized. These chemicals generate additional sludge due to its reaction with suspended particles in the wastewater increasing the total sludge production by an estimated 35%. Ferric chloride sludge will increase the inert solids content and the chemical reaction will also impact alkalinity required for nitrification in the downstream process. The following chemical feed and sludge concentrations were assumed for the mass balance.

- Polymer feed = 1 mg/L
- Ferric chloride = 40 mg/L
- CEPT Sludge Conc. = 40 g/L

CEPT effluent along with HRC effluent and return streams from the backwash clarification process is directed to the intermediate pumping station (IPS). Settled primary sludge is diverted to the sludge blend tank. Refer to Table 5.1 for results. Table B in Appendix E presents conceptual design data for the CEPT process. Table C in Appendix E presents the combined flow and load data to the IPS.

5.2.4 Biologically Active Filtration (BAF)

Flows received at the IPS are pumped to a 2-stage BAF as detailed in the Process Selection Report (Veolia, 2011). The first stage provides carbon removal and simultaneous nitrification-denitrification (NDN). The second stage receives approximately 50% of the NDN effluent and is dedicated for denitrification (DN) with methanol as an external carbon source.

The NDN stage was assumed to have the capability to produce the following effluent quality:

- TSS ≤ 15 mg/L
- cBOD5 ≤ 20 mg/L
- TKN ≤ 5.6 mg/L (based on an effluent ammonia-N of 4 mg/L plus organic nitrogen estimated at 12% of VSS)
- TP ≤ 0.9 mg/L

The Post DN stage was assumed to produce the following effluent quality:

- TSS ≤ 15 mg/L
- cBOD5 ≤ 15 mg/L
- TKN ≤ 5.6 mg/L (based on an effluent ammonia-N of 4 mg/L plus organic nitrogen estimated at 12% of VSS)
- TP ≤ 0.8 mg/L

The final effluent from BAF which consists of a blended of NDN effluent and Post DN effluent is shown in Figure 5.1. Biomass production in BAF cells was estimated at 0.4 g /g BOD5 applied + 0.65 g /g TSS applied per the US EPA Technology Assessment of the Biological Aerated Filter (EPA 600/S2-90/015). Methanol was dosed at 2.45 mg/L to assist denitrification in the Post DN stage and was accounted for in calculating solids generation.

The NDN and Post DN Biofilters requires backwashing to clean the filters and maintain performance. The amount of water (from BAF effluent) required for backwash operations is based on a value of 1575 m3/cell (adopted from PSR, 2011). The backwash waste is then calculated on the basis that the NDN stage consists of 10 cells and the Post DN stage contains 2 cells (information received from John Meunier Inc.). BOD5 removed during backwash of BAF was assumed at 60% of the influent BOD5 load to the BAF. The backwash waste stream from both stages of the BAF is equalized in storage tanks and sent to the existing secondary clarifiers for further treatment. The settling process is chemically enhanced with ferric chloride and polymer addition. The clarified effluent is returned back to the intermediate pump station while the settled sludge is pumped to the solids blend tank. Refer to Section 5.2.5 describing solids handling and treatment. The result of the mass balance is shown in Figure 5.1. Table D and Table E in Appendix E presents conceptual design data for the BAF process. Table F and Table G summarizes the conceptual design data for the backwash waste generated by the BAF. Table H presents the conceptual design data for backwash clarification.

5.2.5 Solids Handling Processes

The sludge handling consists of the following unit processes:

- Sludge Blend Tank
- Thickening
- Stabilization
- Dewatering
- Centrate treatment

The sludge generated from CEPT, HRC and Backwash Clarification processes will be directed to a sludge blend tank. The blended sludge is estimated at 2.7% solids through the mass balance calculations will thickened to a target solids concentration of 5%. A thickener is proposed to reduce the size of anaerobic digesters. Refer to Table I and J in Appendix E for the conceptual design data for sludge blend tank and sludge thickening.

A 2-stage mesophilic anaerobic digestion process is assumed to meet volatile solids reduction criteria for stabilized sludge (biosolids). Following the digestion process it was assumed that the sludge holding tanks would provide a total of seven (7) days of storage prior to centrifuge dewatering. Dewatered biosolids will be temporarily stored in shilos (similar to the NEWPCC dewatering building) from there it will be discharged by gravity to trucks for final disposal.

A preliminary estimation of the anaerobic digester was made using two methodologies which is presented in Table K of Appendix E. Based on mass balance analyses the following concept for anaerobic digestion is suggested:

- A total of four (4) digesters consisting of 2 primaries and 2 secondary.
- Maximum operating volume of each digester is 4.9 ML.
- Primary digesters are estimated at 27.5 m in diameter, completely mixed with fixed cover.
- Secondary digesters are estimated at 27.5 m in diameter, stratified with floating cover.
- Maximum operating depth is 8.2 m.

A suggested mode of operation would be one primary to one secondary digester in a continuous mode. For maintenance, only one digester will be taken down at any given time. The piping network will be set-up to cross either primary to either secondary digester.

The preliminary sizing is conservative and provides a buffer when a digester is taken offline for service. Further analysis is required at the schematic/preliminary design stages should the digester option be carried forward, including a detailed review of gas collection and handling.

Digested sludge is conveyed from the anaerobic digesters by gravity to two (2) sludge holding tanks (SHTs) each approximately 20 m in diameter and 7.3 m in maximum operating depth for a total storage volume of 4.6 ML. This provides about 7 days storage prior to dewatering based on 2031 spring maximum month conditions. From the SHTs, sludge would be pumped to the dewatering process consisting of chemical conditioning and centrifuges. Dewatering was based on operating at 8 hours/d, 7 days a week. Centrate from the sludge digestion and dewatering process is diverted to the centrate treatment process. Dewatered sludge cake is assumed to have a solids concentration of at least 25% which will be easier to haul for disposal. Table L in Appendix E presents conceptual design data for the dewatering process.

Alternative technologies for centrate treatment were not evaluated. In discussion with the Program Team, it was assumed a facility similar to the NEWPCC Centrate Treatment System would be used to reduce effluent TN by 90-95 percent. This includes a Sequencing Batch Reactor (SBR) based process train for nitrification and denitrification with methanol. The final treated effluent can be returned to several possible locations at the head of the plant to allow for phosphorus removal. For this report we propose to return the treated centrate to the headworks to provide a proper blending with the incoming raw wastewater. The optimal return point for the treated centrate will need to be addressed in subsequent engineering analyses.

Based on mass balance analyses, the need for flushing water to reduce the temperature of the centrate stream is not anticipated. However, equalization upstream of the SBR system will be required to allow time for mixing with thickener filtrate to cool the centrate. Downstream equalization is also recommended to buffer the peak decant rate prior to conveying the effluent back to the headworks.

For the mass balance, the centrate treatment system was based on the following:

- Maximum month flow (Spring) = 1.2 ML/d (includes filtrate flows)
- Effluent quality (based on NEWPCC Centrate facility performance, refer to Table 6.8):
- TSS = 88 mg/L
- TKN = 22 mg/L
- TP = 19 mg/L
- Refer to Table M in Appendix E for additional design data for centrate facility.

As stated earlier, Figure 5.1 provides an overall summary of the mass balance calculations. An additional summary table is also provided as Table N in Appendix E. It should be noted the mass balance analyses will be refined as further details are developed in the future design stages.

6.0 Operational Philosophy

The objective of this task was to develop an operational philosophy related to the selective use of existing unit operations and processes, and the addition of BAF, HRC, and wet weather flow disinfection at the SEWPCC based on projected flows and loads to achieve compliance with criteria established by the Program Team. Based on direction provided by the Program Team, Stantec also reviewed the inclusion of side-stream loads (i.e., thickener filtrate and centrate associated with the processing of sludge) in this evaluation. Approximate sizing of the unit operations and processes were estimated based on the peak design flows, and loads that were derived as part of the flows and loads assessment tasks that were previously undertaken as part of the scope of work associated with this assignment (refer to Section 4). This assessment builds and extends upon the previous analyses to provide a quality assurance and verification of the data results of the previous flows and loads assessment, refines the sizing of the new unit processes and flow splits, and provides a more detailed assessment with respect to compliance with established effluent criteria in order to improve the confidence in the overall design and operation of the SEWPCC upgrade and expansion.

6.1 TECHNICAL APPROACH

The objective in this task is to split flows and their corresponding loads to the existing and new unit operations and treatment processes in order to confirm that effluent targets could be realized based on representative performance capacities for the associated unit operations and processes. To accomplish this objective, a stress pattern was developed (as part of an earlier exercise, which was based on actual plant data from 2005 to 2010 inclusive and scaled up based on projected population of 270,000 in the year 2031), and has been routed through the existing and new processes. A detailed numerical “spreadsheet model” was developed and used to simulate treatment alternatives on a mass balance approach to determine if the estimated unit sizing and flow splits will achieve compliance with criteria established by the Program Team, as shown in Table 6.1. Specific scenarios were simulated to assess compliance for various flow splits and unit sizes.

Table 6.1 – Effluent Compliance Requirements

Parameter	Averaging Period	Limit	Units
Total Suspended Solids (TSS)	30-day rolling average	≤ 25.0[a]	mg/L
5-day Carbonaceous Biochemical Oxygen Demand ($cBOD_5$)		≤ 25.0[a]	mg/L
Total Phosphorus (TP)		≤ 1.0[a]	mg/L
Total Nitrogen (TN)		≤ 15.0[a]	mg/L
Ammonia Nitrogen - January	Daily never-to-exceed	≤ 1,975[a]	kg/day as N
Ammonia Nitrogen - February		≤ 2,403[a]	kg/day as N

Parameter	Averaging Period	Limit	Units
Ammonia Nitrogen - March		≤ 4,196[a]	kg/day as N
Ammonia Nitrogen - April		≤ 12,926[a]	kg/day as N
Ammonia Nitrogen - May		≤ 5,311[a]	kg/day as N
Ammonia Nitrogen - June		≤ 3,103[a]	kg/day as N
Ammonia Nitrogen - July		≤ 1,517[a]	kg/day as N
Ammonia Nitrogen - August		≤ 607[a]	kg/day as N
Ammonia Nitrogen - September		≤ 703[a]	kg/day as N
Ammonia Nitrogen - October		≤ 811[a]	kg/day as N
Ammonia Nitrogen - November		≤ 1,152[a]	kg/day as N
Ammonia Nitrogen - December		≤ 1,550[a]	kg/day as N
Ammonia Nitrogen - Year-round: Lethal to fish	Never-to-exceed	≤ 50%[b]	fish mortality
E-coli and Fecal coliform	30-day geometric mean	≤ 200.0[c]	MPN/100 mL

*Notes: a – 24 hour effluent composite sample
b – 96 hour static acute lethality test, pH adjusted
c – grab sample collected at equal time interval on each of a minimum of 3 consecutive days per week

It is important to note that the stress pattern represents the flows and loads that will likely be experienced in the year 2031 based on projected growth and additional wet weather flows from new developments in the SEWPCC service area, assuming that the habits, sector composition, and collection system behavior and response remain comparable to existing conditions. There is always the possibility that projected flows and loads could over-or underestimate the actual flows and loads. In addition, the performances selected for the various unit processes and operations are typical design capacity values, and should be validated in subsequent engineering phases to confirm that the values are appropriate for the SEWPCC. Specifically, based on the selected performance values and estimated processes sizes, simulations will help confirm that that the sizing of the unit processes are within an acceptable range for subsequent engineering design purposes

The selection of an appropriate design population for plant expansions and performance capacities for unit operations and processes are critical factors influencing size requirements, and associated capital and operating costs. A 20-year design horizon balances the goals of not building too far in advance of utilization against the negative impact of frequent construction activities on operations. More importantly, there is uncertainty in the timing when the design population of 270,000 people is reached. If growth occurs more rapidly than expected, the capacity of the facility will be exceeded earlier than projected, and will require expansion earlier than planned. Conversely, if growth does not materialize as quickly as forecasted, there will be capital investment in a facility that is underutilized for more years than planned and this condition may result in operational difficulties. With this understanding and limitations, a stress pattern was developed and used to confirm that the sizing of the BAF and HRC were within an acceptable range, based on flow and load splits and predicted compliance with effluent criteria for subsequent engineering design purposes.

The following sections elaborate on the technical approach, organization of the data, use of representative performance capacities for the unit processes, and compliance assessment for various processes sizes for specific flow split scenarios.

6.2 SYSTEM CONFIGURATION

Based on projected flows and loads for a population of 270,000 people in the year of 2031 (refer to Table 4.18), and direction received from the Program Team to reuse to maximum extent possible the existing assets, the upgrade and expansion of the SEWPCC was defined to consist of an optimal sizing and configuration of the following unit process and operations:

- Pumping, screening, and grit removal
- Reuse of exiting primary clarifiers and conversion to CEPT
  - It is important to note that the existing primary clarifiers will no longer receive waste activated sludge (WAS) when the system is converted to a BAF process and will correspondingly result in an increase in the clarification performance capacity
  - Addition of appropriate chemicals to improve the coagulation, flocculation and sedimentation of raw sewage and phosphorus removal
- Possible expansion of the existing primary clarifiers and/or the addition of supplemental HRC for dry weather flows
- Inclusion of a BAF for carbon and nitrogen removal
  - Sizing dependent on achieving compliance with effluent targets
  - Capacity and performance of the BAF to account for the internal backwash flows and loads
- Reuse of the existing secondary clarifiers for clarification of backwash waste
- Inclusion of supplemental HRC
  - Sizing dependent on achieving compliance with effluent targets
  - HRC to handle excess wet weather flows and loads that are not routed through the BAF

- Combination of BAF and HRC to be allow treatment up to the projected peak day flow of 325 ML/d

- Expansion of the existing UV disinfection system to match the maximum treatment capacity of the BAF

- Addition of a chlorination and de-chlorination facility to disinfect flows that are not disinfected by the expanded UV treatment facility, up to the projected peak day flow of 325 ML/d

- Possible addition of metal salts in the clarification, digestion, and centrate treatment processes to effectively reduce the phosphorus content of the treatment streams to within compliance limits.

- Inclusion of digestion, dewatering, and centrate treatment on site:

  - A common holding tank to be used for the collection and storage of sludge from the primary clarifiers, HRC, and backwash clarification

  - Filtrate from the holding tanks to be sent to the head end of the plant

  - Treated centrate to be sent to the head end of the plant, CEPT, or to the intermediate pumping station ahead of the BAF. Strong consideration should be given to sending treated centrate to the head end of the plant because it can provided certain process benefits (i.e., nitrates can be used to reduce odor and chem-P removal via CEPT). For clarity reasons and consistency with the remainder of the validation assessments, it has been assumed that treated centrate liquid will be sent to the head end of the plant downstream of screening and grit removal, and centrate WAS will be sent to the sludge thickening and blend tank.

The design philosophy will need to take into consideration the operational requirements during periods when a unit process needs to be taken off-line for maintenance reasons. Specifically, to achieve full compliance on a year-round basis, the design may need to have an additional unit process or operation for critical treatment trains. This will allow a unit to be taken out of service and still be able to provide full rate treatment performance and compliance with effluent design criteria.

The optimum sizing of the unit processes to meet effluent compliance criteria is strongly influenced by flow and the load split to the specific treatment process and whole life costs associated with the upgraded or expanded process. A representative schematic of the system expansion and upgrade, along with the configuration of the processes is shown in Figure 6.1. It is important to note that the process configuration for compliance assessment purposes contains a chemical based system for P removal. The Project Team has requested that alternative site layouts be developed so that P can be removed biologically in the future. With this direction, Stantec reviewed possible process configurations so that the conversion from a chemical-based to biological-based P removal system would be possible and that sufficient space was allocated for its future implementation. As part of the review it was identified that the HPO tanks would need to be retrofit to provision the appropriate zones (i.e., anaerobic and aerobic) and that functionality of the existing secondary clarifier would need to be preserved to facilitate biological phosphorus removal.

The process evaluation and design of a biological-based P removal system is beyond the scope of this assignment. The objective of this request was to conceptualize a process configuration that would support a biological based P removal system, maximize as much as possible the reuse of existing and new assets associated with a BAF based nitrogen removal system in concert with a chemical-based P removal system and provision sufficient space on site to logically integrate a biological-based P removal system.

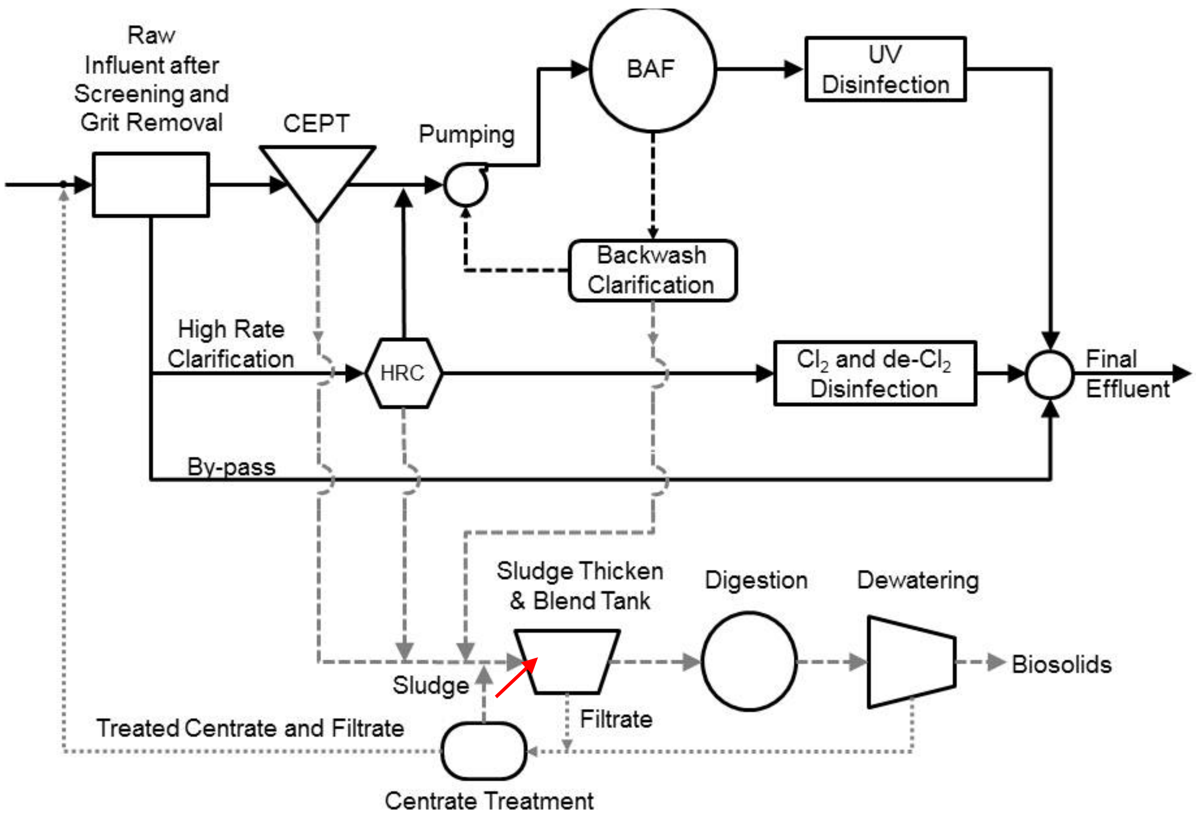


Figure 6.1: Schematic of Upgraded and Expanded SEWPCC Processes and Unit Operations

As a starting point, the processes were sized to achieve compliance as best as possible with specific parameters and limits given in Table 6.1 based on projected 2031 peak flows and loads identified in Table 4.18 for maximum week summer conditions. Specifically, the sizing of the BAF and HRC were based on the information contained in Table 4.18 and an independent compliance assessment was done using a stress pattern to more accurately predict the performance of an integrated system, especially for diurnal variation on an hourly basis. It was assumed that a BAF treatment technology was capable of reliably achieving an effluent ammonia concentration of 1.5 mg/L-N based on discussions with suppliers. Preliminary assessment indicated that at least 200 ML/d would need to go through the BAF, and 325 ML/d would need to be clarified through a combination of CEPT and HRC to achieve an effluent ammonia concentration of 2.8 mg/L L-N for maximum week summer loading conditions to comply with ammonia effluent loading restrictions. For example,

Given conditions:

- Maximum allowable effluent load limit for August: 607 Kg/d
- Maximum week flow summer: 220 ML/d
- BAF capable of achieving 1.5 mg/L -N based
- Summer ammonia concentration ~ 14 mg/L -N based on ammonia to TKN ratio of 0.62

Assessment:

- 200 ML/d * 1.5 mg/L + 20 ML/d * 14 mg/L = 580 Kg/d
- Treating 200 ML/d via BAF and shunting 20 ML/d to HRC (assumes no reduction in raw influent ammonia concentration) will achieve compliance with August ammonia effluent load limits on a maximum summer week basis.

To achieve full compliance on a maximum day basis, it would require the BAF system to treat 325 ML/d. Subsequent modeling would more accurately assess ammonia excursions for a 200 ML/d BAF based treatment process.

It was found that compliance with the ammonia limits and associated averaging period created the greatest challenge and would need greater focus and effort to determine practicable and cost-effective design trade-offs. It was assumed that the existing primary clarifiers could be converted, expanded and operated in CEPT mode to treat flows up to 150 ML/d. The HRC was sized to treat flows beyond the CEPT limit up to 175 ML/d, to provide a peak day clarification capacity of 325 ML/d. Peak hour flows in excess of 325 ML/d up to 420 ML/d would receive screening and grit removal and then be bypassed around the clarification chemical/biological treatment processes. A sample flow logic scheme is outlined below and provided in schematic form in Figure 6.2:

- All flows up to 420 ML/d will receive screening and grit removal
- Flows up to 200 ML/d will receive primary treatment through a combination of CEPT (i.e., 150 ML/d) and HRC (i.e., 50 ML/d) and will be sent to the BAF for treatment and conveyed to the UV disinfection. This implies that the UV treatment needs to be sized to match the flow and treatment capacity of the BAF

- Flows greater than 200 ML/d and up to a maximum of 325 ML/d will receive primary treatment through HRC followed by chlorination and dechlorination. This implies that the wet-weather disinfection system will need to treat up to 125 ML/d
- Peak hourly flows beyond 325 ML/d will be by-passed around primary treatment, secondary treatment and disinfection
- All treated and by-passed flows will be blended as determined by the flow logic noted above

Since whole life costs are outside of the scope of work in the assignment to Stantec, it is not possible to determine the most cost-effective process configuration and trade-offs between process sizes.

6.3 DEVELOP SYNTHETIC STRESS PATTERN

6.3.1 Introduction

Development of a stress pattern was required to allow assessment of the performance of an integrated system of unit processes and operations to achieve compliance with criteria provided by the Program Team. Based on direction provided by the Program Team, Stantec also reviewed the inclusion of side-stream loads (i.e., thickener filtrate and centrate associated with the processing of sludge) in this evaluation. Approximate sizing of the unit operations and processes were estimated based on the peak design flows and loads that were derived as part of the flows and loads assessment (refer to Section 4.2 and 3.3). This assessment builds and extends the previous work and analyses to provide a quality assurance and confirmation of the data results of the previous flows and loads assessment. The criteria provided by the program team defines the effluent quality the treatment system (i.e., CEPT+HRC+BAF+ disinfection) must achieve for projected flows and loads to be in compliance.

6.3.2 Compliance Technical Analyses

The purpose of this exercise was to develop a representative future 20-year loading pattern to assess the performance and sizing requirements of upgrades and expansions at the SEWPCC to achieve compliance with criteria established by the Project Team. The loading pattern represents the process treatment stresses that will likely be experienced in the future in response to growth in the SEWPCC service area. The “stress pattern” represents the best approximation of flows and loads the upgrade and expanded plant will experience based on historic patterns that have been experienced at the SEWPCC over the past six years (2005 to 2010, inclusive). The flows and loads were divided into hourly time steps because sustained peak hour bypasses around the BAF and/or the HRC has the potential to increase the actual load and concentration of certain parameters in the final effluent, especially ammonia. Currently, flow proportioned samples are collected once per hour at the SEWPCC to establish a 24-hour daily composite sample. The use of the average daily flow attenuates the diurnal and/or peak wet weather flow variations. As such, it is important to quantify hourly bypasses around certain processes and to determine if they are of sufficient quantity to impact the final 24-hour composite sample, which could result in non-compliance with established criteria.

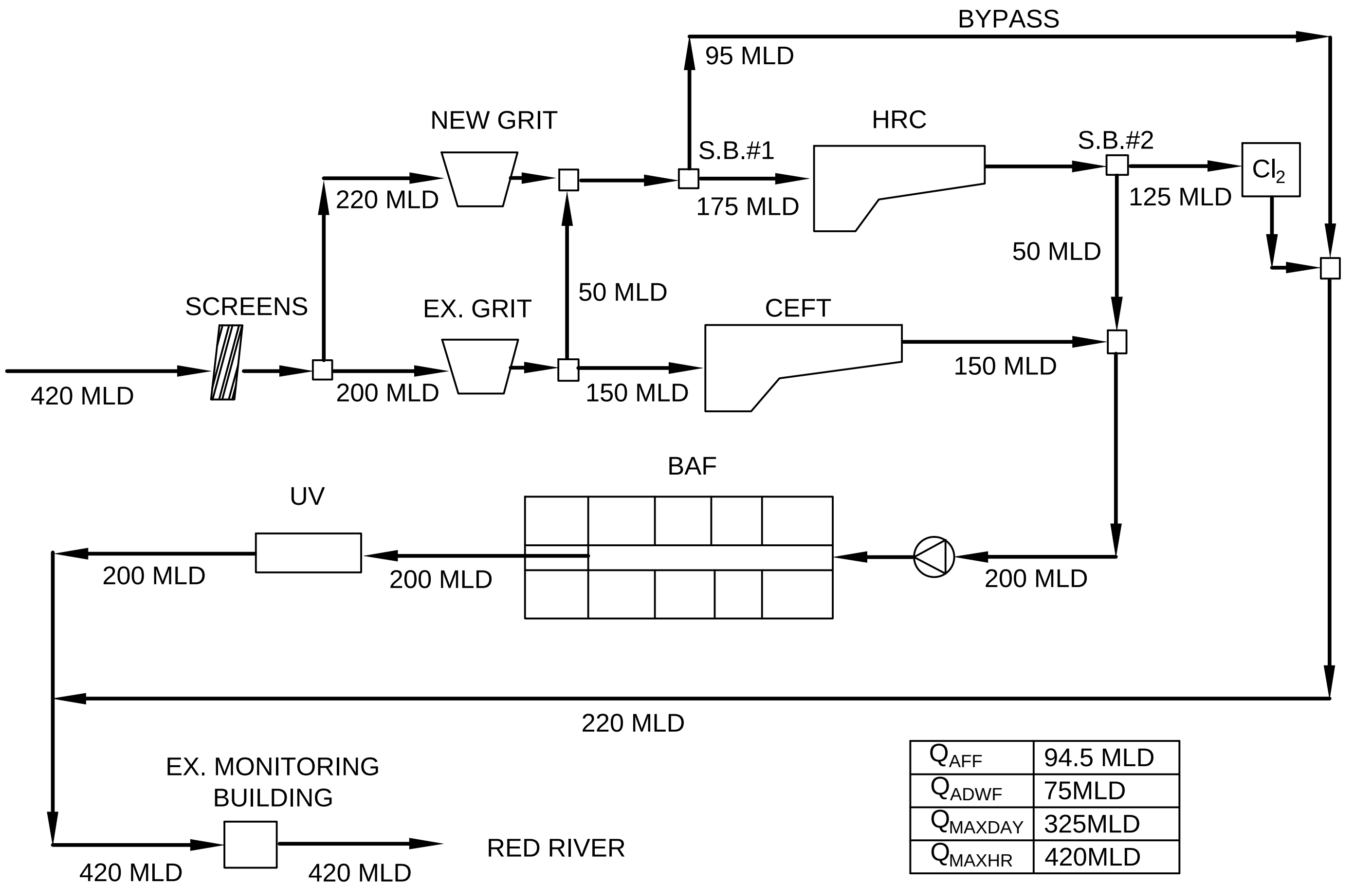
The Project Team requested that that the future flows and loads consider that the Windsor Park sewer district would flow year-round to the SEWPCC, and that hauled liquid waste would no longer be accepted at the SEWPCC. These requirements were included in the projected future flows and loads for the year 2031.

There is always the possibility that projected flows and loads for the 20-year design horizon could over or underestimate the actual flows and loads for a variety of reasons, (e.g., more or less: population for the year 2031, businesses and industries locate within the SEWPCC service area, wet weather flows/climate change, or changes in the collection system). With this understanding, the last six years were used as the basis for the future flows and loads and scaled up based on expected service area population in the year 2031 and an allowance made for extraneous wet weather flows from new development.

6.3.3 Load Projections

The loads to the SEWPCC (e.g., solids, cBOD, ammonia, TKN, phosphorus, and other parameters) were assumed to be directly proportional to the service population. It was assumed that the current fractionation for each parameter would remain similar for future loads. An increase from the average population of 184,000 for the past five years (i.e., 2006 to 2010, inclusive) to a projected population of 270,000 people in the year 2031 represents an increase of about 147% in the expected load to the SEWPCC for the year 2031. It was also recognized that certain parameters might experience and increase in load related to flow due to the fact that some of the sewer districts have combined sewers and that some addition load from street wash off could occur. As such, each parameter was reviewed to determine if a correlation between load and flow existed. As expected, some parameters, such as total suspended solids (TSS) did exhibit a load increase with wet weather flows, while others such as ammonia load were virtually insensitive to increasing flow. As part of this review it was found that one TSS data point was suspect and was subsequently removed from the dataset to prevent an artificially high skewing of the TSS load.

A review of the septage loads hauled to the SEWPCC was performed to determine its flow and load contribution. The review found that the flow contribution from HLW was extremely low in comparison to the flows conveyed via the St. Mary's Road interceptor. Based on the HLW data provided, the maximum volume received at the SEWPCC (May 7, 2008) was 473 KL or 0.473 ML. Assuming this volume is received during normal working hours over an 8-hour period would be the equivalent of about 1.42 ML/d, this extreme single event would represent about 2.8% to the total flow. Normal HLW volumes are in the order of 40 KL per day which represents about 0.24% of the winter dry weather flow. As such, the flow contribution and influence is small enough that it can be ignored without any influence on plant flow hydraulics at this point in the design. The only significant load contribution from HLW relates to TSS, and primarily from non-household sources. The removal of HLW acceptance from the SEWPCC was discussed in Section 4.3.1. Based on a population of 184,000 people and an equivalent daily load reduction of 0.004 kb/person/day, the base daily load would drop by 736 Kg/d. Accordingly, all historical TSS values used in the stress pattern were discounted by 736 Kg/d to account for the removal of HLW from future loads to the SEWPCC.



Q_{AFF}	94.5 MLD
Q_{ADWF}	75MLD
Q_{MAXDAY}	325MLD
Q_{MAXHR}	420MLD

Stantec Consulting Ltd.

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Tel 204-489-5900 Fax 204-453-9012

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DATE	DATE

ENGINEER'S SEAL

TENDER NO.

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

PRELIMINARY FLOW SCHEMATIC
FOR 2031

CITY DRAWING NUMBER: FIGURE 6.2

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There was insufficient information to include a “flush phenomena” in the stress pattern, even though it is known to occur when a sufficient amount of extraneous water enters the collection system and induces sufficient scouring velocity to re-suspend.

It was assumed that scaling up loads based on population would be representative of loads received at the SEWPCC in the future. This assumption is based on the fact that the vast majority of the sewer service area is served by separate sewer systems for wastewater and land drainage. Some areas are serviced by a combined sewer (CS) system, which can add some additional constituent loads into the system from street wash off. The amount of rainwater and flows associated with street wash-off tends to significantly dilute the concentrations of sewage. The combined sewer carries in the order of 40 to 100 times the normal dry weather flow. Over time the diversion structures which were designed to convey about 2.75 times the dry weather flow to the treatment plants have been raised and capture a greater amount for conveyance to the SEWPCC for treatment. It is important to note that the pumping and conveyance systems associated with the CS districts limits the amount of flow that is directed from those districts to the SEWPCC. As such, it has been assumed that wet weather flows greater than 2.75 times the dry weather flow are spilled to the Red River. The maximum mass conveyed occurs at incipient overflow. Flows greater than 2.75 times the DWF are spilled along with its proportion of mass loading. As such, the larger the wet weather event, the greater the loss of mass loading from the CS districts.

Typically, a rainfall of 5 mm will cause a run-off equivalent to about 3 mm of rainfall. These small events do not wash any significant amount of street related matter in the system. A rainfall of 8 mm will result in about half of the mass of sewage-related matter to be spilled to the river. In essence, the greater the rainfall event beyond 5 mm, the greater the loss of sewage related mass from the system. This condition is evidenced in Table 4.19, i.e., lower concentrations associated with flow weighed averages. The Mager sewage lift station has a total capacity of 44.6 ML/d and represents about 11.7% of the total flow and load to SEWPCC. As the population increases in the SEWPCC service area the percentage will decrease. In addition the new developments are less prone to wet weather inflows. As such, less of the sewage load will be lost from these new areas in the form of emergency overflows. Two of the CS districts, Mager and Cockburn, are being upgraded for purposes of enhanced basement flood protection. This will result in less wet weather intrusions into the system and possible loss of sewage to the rivers.

The foregoing discussion supports that a small proportion of additional matter enters the combined sewer system via street wash-off that makes it to the SEWPCC and will diminish with sewer system upgrades, new developments will experience less wet weather intrusions and likelihood of emergency overflows, and two of the existing combined sewer districts will contribute less wet weather flows in the near future. More importantly, the vast majority of matter entering the sewer system is from spent waters associated with human-related activities and will increase in direct proportion to the population.

The wet weather flows entering the system in separate sewer areas tend to be relatively clean water which tends to dilute the wastewater constituent concentrations. Since the sewers typically flow less than a third full, suspended matter tends to settle in the sewers and is flushed when sufficient flows and velocities are achieved, typically during wet weather events. This is referred to as the “first flush” or the flush phenomena due to wet weather flows. Diurnal sampling during wet weather events would help to better characterize the loading variation experienced at the SEWPCC and used to optimize treatment processes. It is recommended that diurnal sampling of the raw influent be conducted to provide wet weather loading data for process optimization.

6.3.4 Parameter Data for Stress Pattern

As shown in Table 6.2, the City of Winnipeg has collected the following influent data, to greater and lesser degrees:

Table 6.2 - Summary of Influent Data Collected at the SEWPCC

pH	Nitrite + Nitrate (NO_2+NO_3)
Flow, ML/d	Total Kjeldahl Nitrogen (TKN)
Temperature, °C	Total ammonia (NH4)-N
Alkalinity, mg($CaCO_3$)/L	Ortho Phosphorus, (P_{sol})-P
Total Suspended Solids (TSS)	Total Phosphorus, (TP)-P
Volatile Suspended Solids (VSS)	Total Organic Carbon (TOC)
Uninhibited, Biochemical Oxygen Demand (BOD)	Chemical Oxygen Demand (COD)
Inhibited, Carbonaceous Biochemical Oxygen Demand (cBOD)	Total Volatile Fatty Acids (VFA)

The City began collecting addition data in 2007 in response to recommendations made by Stantec in 2006 to better characterize certain water quality parameters and to improve the confidence in the wastewater characterizations for design purposes. The current review and analyses of flows and loads has the benefit of this additional data. Where data was missing from the historic data set, it was populated based on trends that were observed in the available data (January 1, 2005 to June 30, 2011, inclusive). This was done by plotting the available load data against flow and performing a simple statistical analysis and regression on the data, followed by insertion of data that was consistent with past trends and flow regime. From these analyses, it was possible to develop relationships between the parameters and to fully populate a data set for the following parameters to develop loads for a stress pattern.

The following graphs, Figures 6.3 to 6.7, summarize the analyses performed on the data, and presents the correlations with the equations used to infill missing data.

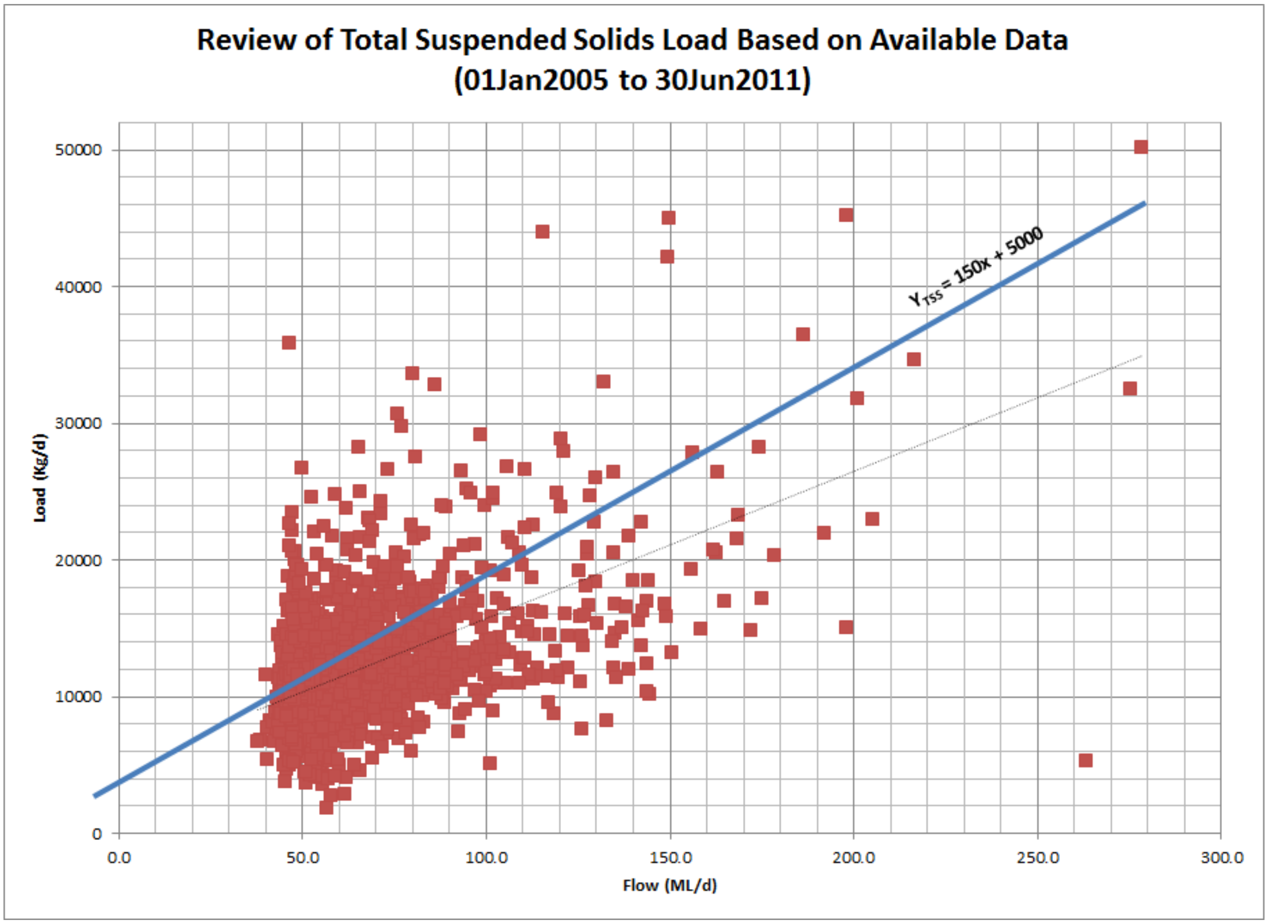


Figure 6.3: Trend and Correlation Analysis for TSS

- TSS_{LOAD} = 150 x flow + 5,000

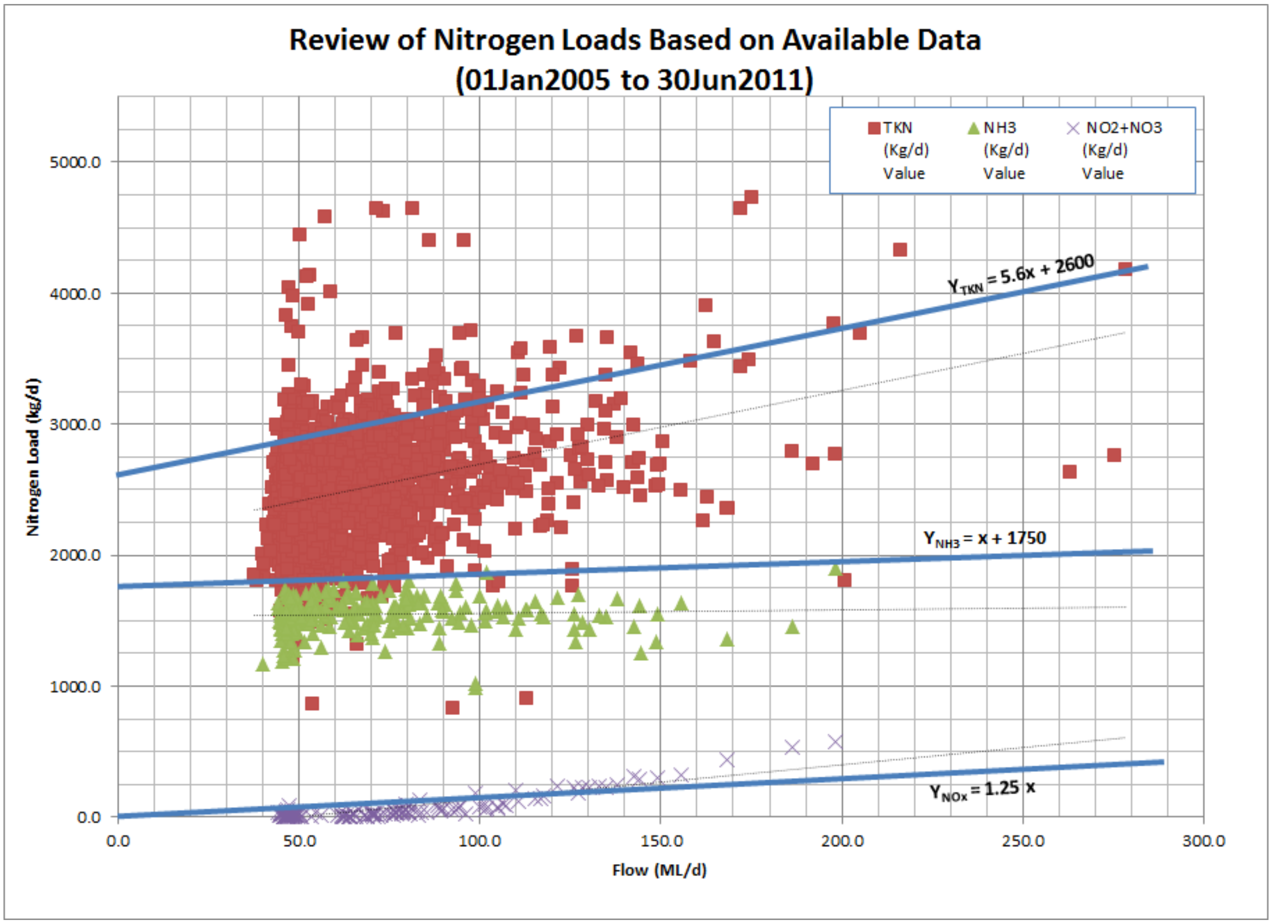


Figure 6.4: Trend and Correlation Analysis for Nitrogen

- TKN_{LOAD} = 5.6 x flow + 2,000
- $NH3_{LOAD}$ = flow + 1,750
- NOX_{LOAD} = 1.25 x flow

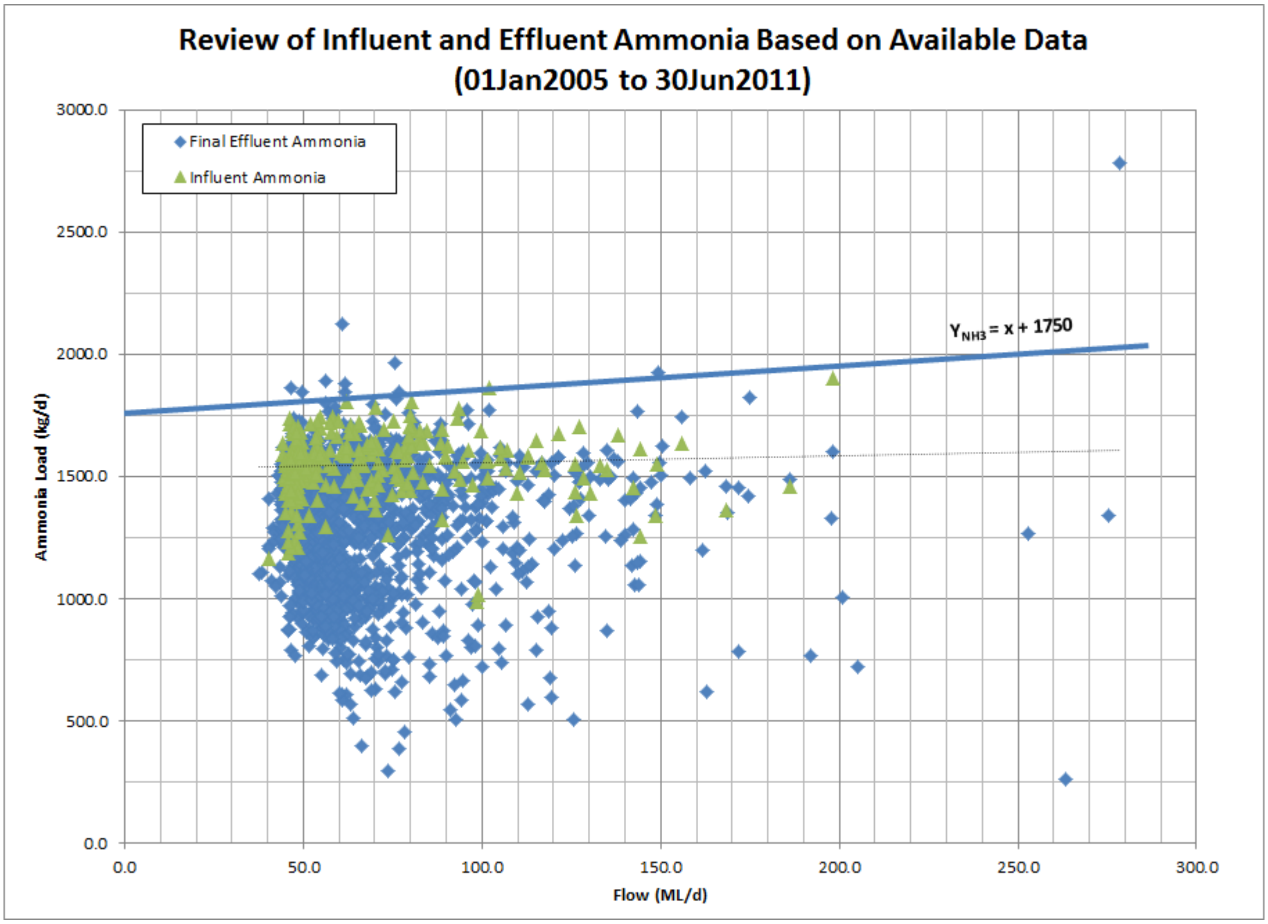


Figure 6.5: Trend and Correlation Analysis for Effluent Ammonia

- $NH3_{LOAD}$ = flow + 1,750

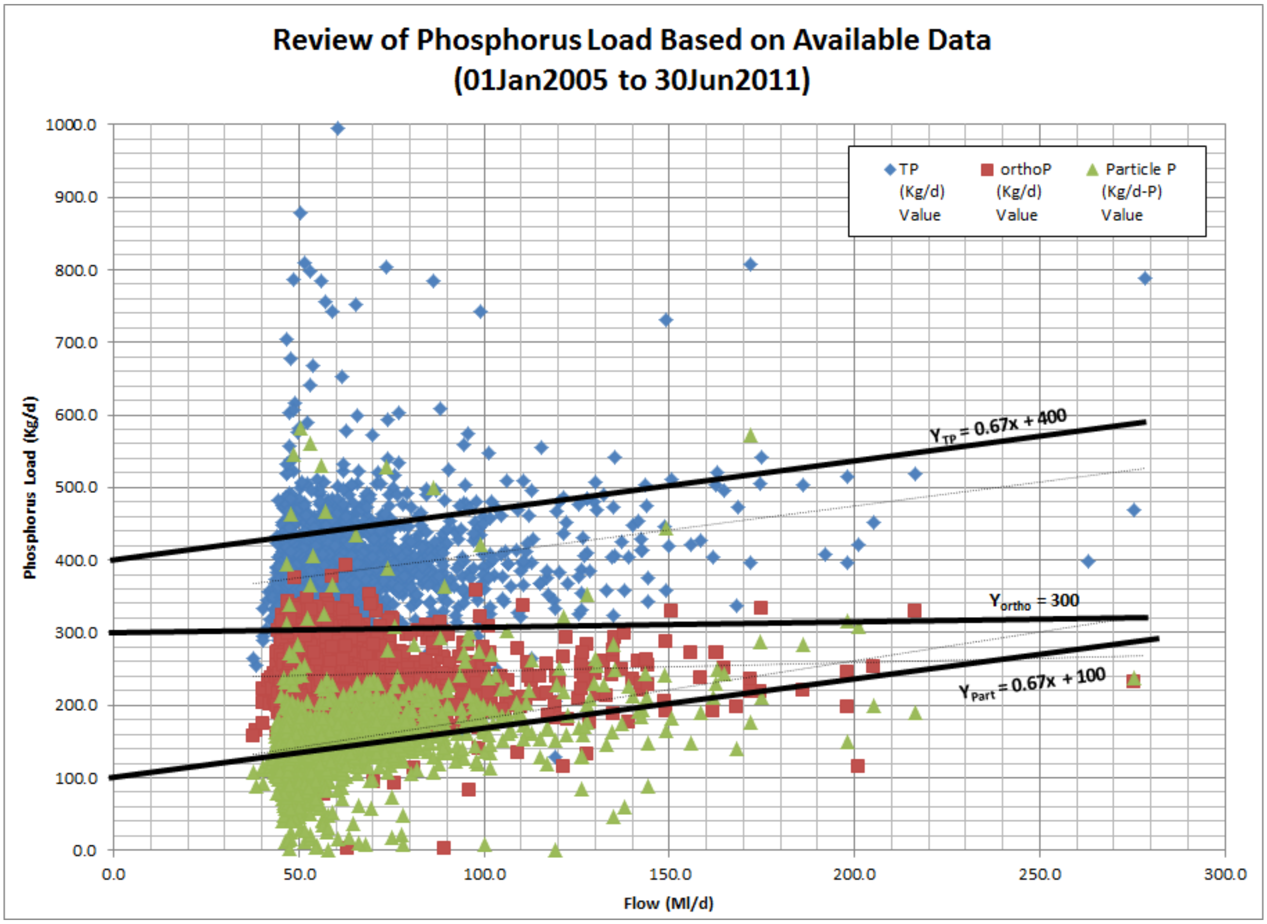


Figure 6.6: Trend and Correlation Analysis for Phosphorus

- $PTOP_{LOAD}$ = 0.67 flow + 400
- $PSOL_{LOAD}$ = 300
- $P.Part_{LOAD}$ = 0.67 x flow + 100

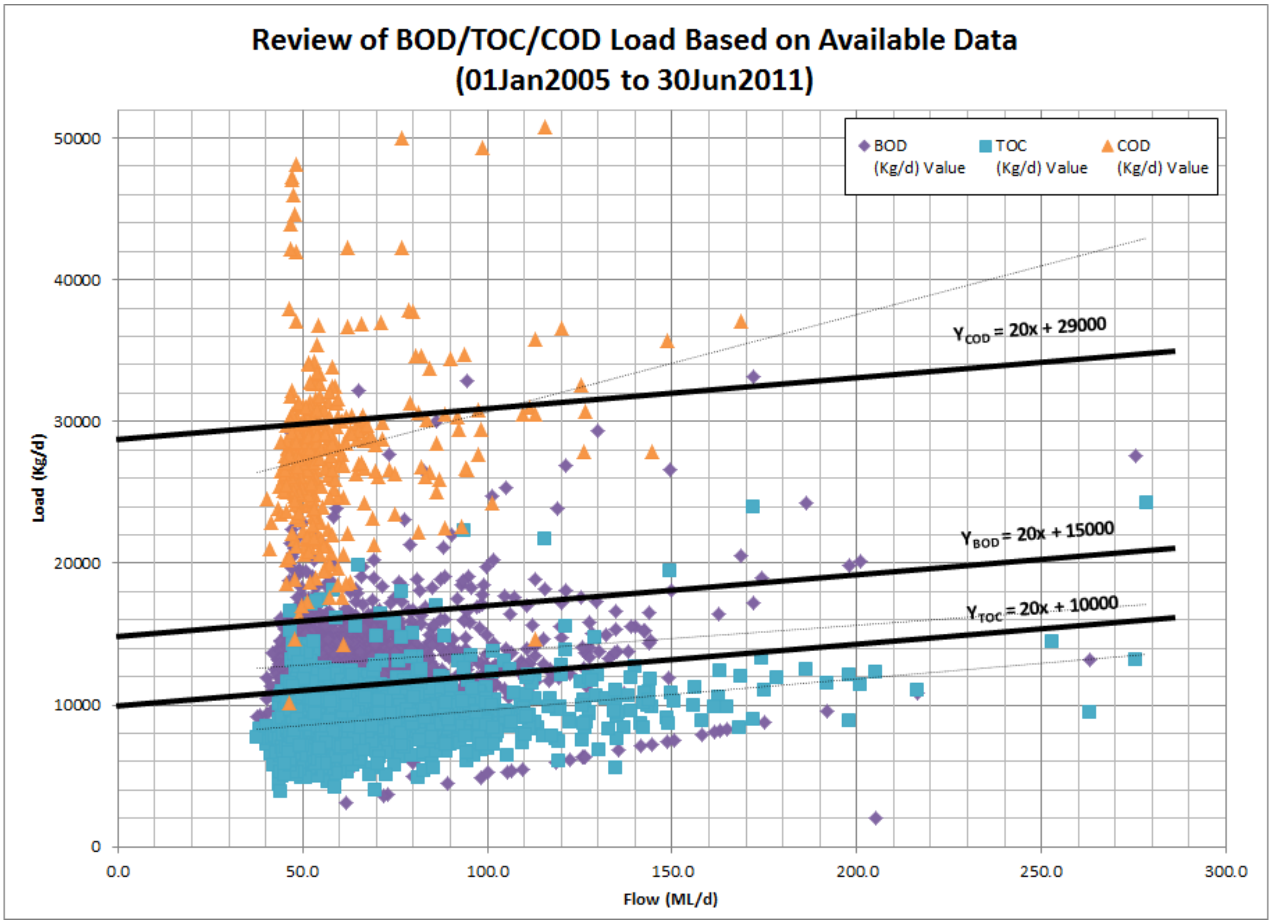


Figure 6.7: Trend and Correlation Analysis for BOD/TOC/COD

- COD_{LOAD} = 20 x flow + 29,000
- BOD_{LOAD} = 20 x flow + 15,000
- TOC_{LOAD} = 20 x flow + 10,000

6.3.5 Flow Patterns

A review was conducted on the flow patterns received at the SEWPCC to partition the flows into dry and wet weather flows. This review suggested that flows below 70 ML/d were considered to be within the normal dry weather flow range. Flows above 70 ML/d were assumed to have a wet weather component associated with the overall flow. Based on detailed flow records (January 1, 2005 to June 30, 2011, inclusive) collected in increments of minutes, it was possible to construct hourly diurnal flow patterns for the historic data. The diurnal flows were then scaled up in direct proportion to the increase in population, such that the current flows were scaled up by factor of 147%.

A review of the hourly data was performed to develop a representative diurnal pattern for new wastewater flows associated with the additional population. The historical data that was provided in the form of either 1 minute or 6 minute increments were consolidated into hourly data to develop these patterns. It was found that the month of December (i.e., 12) shows the greatest variation over the day and was adopted for use to develop an hourly pattern for new flows associated with additional population. The new flows were superimposed on top of the existing hourly flows to develop a composite diurnal pattern. The review is summarized in Table 6.3, and graphically displayed in Figure 6.8.

Table 6.3 - Ratio of hourly flow to Daily Average

	Month											
Hour	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
0	1.11	1.10	1.07	1.09	1.08	1.07	1.06	1.08	1.08	1.08	1.09	1.08
1	1.03	1.01	0.99	1.01	1.01	1.03	1.03	1.01	1.00	0.99	0.99	1.00
2	0.92	0.90	0.88	0.91	0.92	0.93	0.96	0.91	0.91	0.89	0.89	0.90
3	0.80	0.78	0.78	0.83	0.82	0.83	0.88	0.81	0.79	0.78	0.78	0.78
4	0.67	0.64	0.67	0.75	0.73	0.75	0.80	0.71	0.69	0.68	0.67	0.66
5	0.57	0.57	0.59	0.70	0.68	0.69	0.72	0.64	0.62	0.59	0.58	0.59
6	0.51	0.51	0.55	0.68	0.65	0.67	0.69	0.61	0.60	0.55	0.54	0.55
7	0.52	0.52	0.57	0.69	0.68	0.70	0.69	0.63	0.64	0.58	0.58	0.55
8	0.62	0.61	0.64	0.76	0.76	0.80	0.75	0.70	0.71	0.66	0.66	0.62
9	0.74	0.78	0.76	0.87	0.91	0.94	0.87	0.83	0.87	0.84	0.82	0.72
10	0.92	0.97	0.93	1.00	1.04	1.05	1.00	0.99	1.04	1.02	1.01	0.91
11	1.10	1.15	1.07	1.08	1.15	1.15	1.11	1.12	1.19	1.19	1.16	1.08
12	1.22	1.25	1.14	1.14	1.19	1.18	1.17	1.21	1.27	1.25	1.24	1.20
13	1.29	1.30	1.19	1.14	1.20	1.19	1.19	1.24	1.28	1.26	1.28	1.29
14	1.29	1.28	1.21	1.14	1.19	1.15	1.17	1.24	1.25	1.25	1.29	1.35
15	1.25	1.22	1.19	1.14	1.16	1.15	1.15	1.21	1.18	1.21	1.25	1.33
16	1.20	1.19	1.19	1.11	1.10	1.09	1.11	1.16	1.11	1.17	1.16	1.27
17	1.17	1.15	1.22	1.11	1.07	1.06	1.08	1.14	1.08	1.13	1.13	1.18
18	1.18	1.15	1.22	1.12	1.09	1.07	1.08	1.12	1.07	1.12	1.11	1.16
19	1.17	1.16	1.26	1.12	1.08	1.07	1.10	1.13	1.09	1.12	1.14	1.17
20	1.19	1.19	1.24	1.15	1.10	1.09	1.11	1.14	1.12	1.15	1.16	1.16
21	1.21	1.21	1.26	1.16	1.12	1.11	1.11	1.14	1.14	1.18	1.17	1.19
22	1.19	1.21	1.21	1.15	1.14	1.10	1.10	1.13	1.15	1.19	1.17	1.15
23	1.15	1.16	1.19	1.13	1.13	1.10	1.08	1.11	1.12	1.15	1.13	1.11

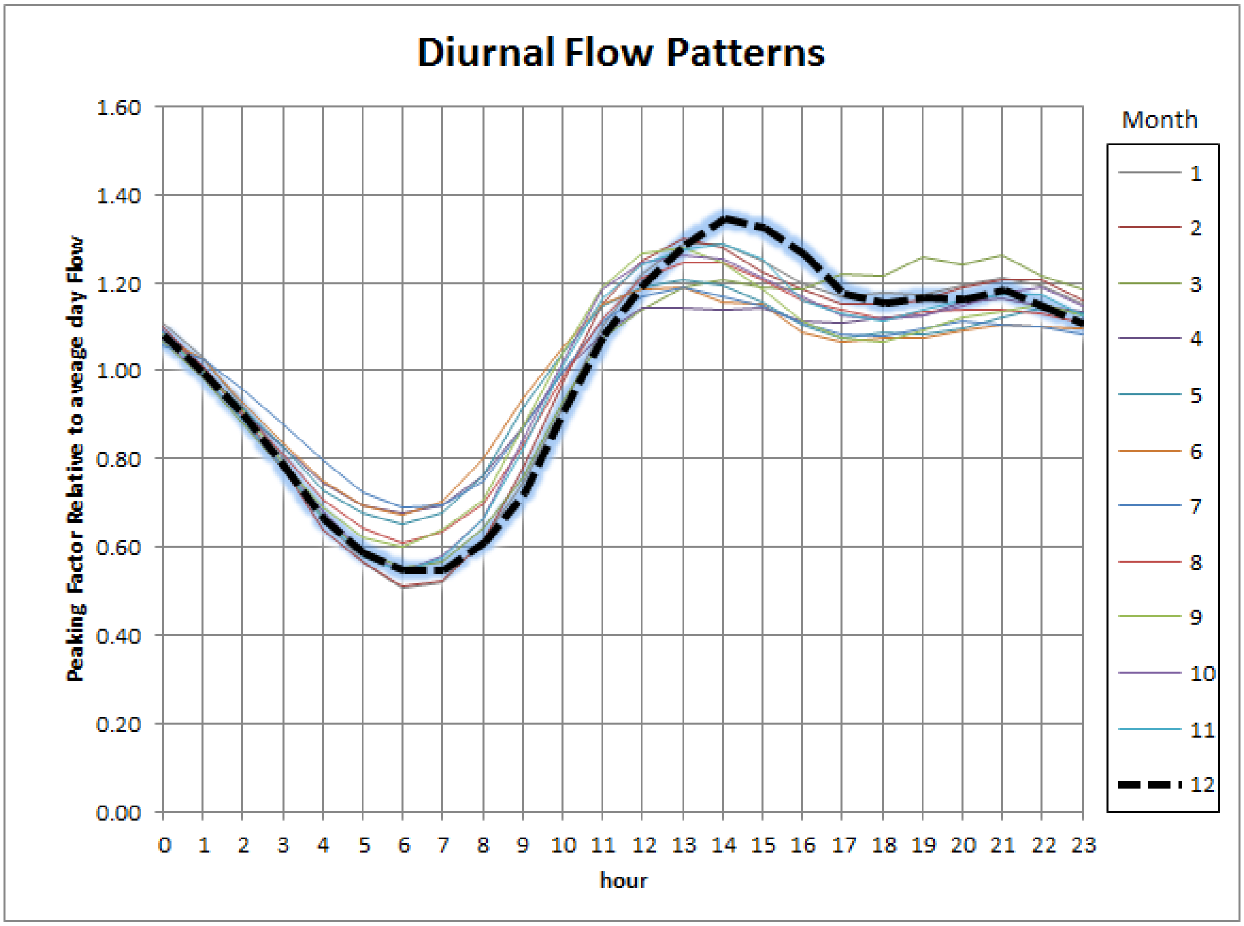


Figure 6.8: Month-specific Average-day Diurnal Flow Pattern at the SEWPC

Flows greater than 70 ML/d were partitioned into a wet weather bin. New developments were assumed to have less extraneous inflows, and a factor of 50% of wet weather contribution from existing areas was used to allow for extraneous wet weather flows from new developments. As such, new developments were scaled up in population at 50% to generate a wet weather factor of 123%. As such, flows greater than 70 ML/d were scaled up by 123%.

The scaled-up dry and wet weather flows were recombined to generate a projected hourly 2031 flow pattern.

6.3.6 Projected 2031 Stress Pattern

Existing loads were assumed to be directly related to population, with additional loads accounted for based on wet weather load trend analyses. The existing loads were then scaled up in direct proportion to the population increase, irrespective of flows. The projected loads were divided the projected flows to derive an equivalent 24-hour composite sample concentration for the year 2031. It was assumed that the concentration would to be constant over a 24-hour period for this analysis. Specifically, the equivalent 24-composite concentration for any given day was multiplied by the flow on the same day which then represents the total load for that day.

Applying this approach to the scaled up flows and loads, it was possible to construct a stress pattern that mimics the patterns of dry and wet weather flows and loads observed over the past five years at the SEWPCC. This stress pattern provides a logically scaled up 6-year period of flows and loads in discrete hourly increments to model process performance and predict the performance and compliance for different combinations/efficiencies and sizes of treatment units.

While the flows and loads developed earlier in this section were used to estimate the size of unit operations and treatment processes, the stress pattern is applied to simulate flows and loads to and through the upgraded plant to assess effluent compliance for various flow splits and representative performance expectations for specific treatment processes. The compliance requirements and their associated averaging periods, especially during wet weather conditions, will affect the sizing of the treatment units based on expected performance and how much diluted flow and loads can be blended with treated effluent, while still achieving compliance. Since compliance with effluent ammonia is based on a never-to-exceed daily basis for month specific load limits, the blending of flows beyond the BAF treatment capacity is a key design consideration. Because of the high concentration of pathogens in raw influent the amount that can be blended along with the level of disinfection required to achieve compliance during wet weather conditions will influence the flow splits and treatment performance requirements. Lastly, the proposed 98% compliance with $cBOD_5$ and TSS to achieve 25 mg/L or less in the final effluent essentially translates to a never-to-exceed basis. This will require a review of the sizing and split between treatment of dry and wet weather flows, and the wet weather flow treatment that can be blended to achieve a never-to-exceed limit of 25 mg/L for both TSS and $cBOD_5$.

Since compliance with ammonia, E.coli, TSS and $cBOD_5$ is related to the amount of wet weather flows receiving treatment and the amount of dilute raw influent being blended in with treated effluent, it is important to understand how peak hourly wet weather flows and loads can affect the daily loads. Specifically, diurnal variations and the peak and duration of wet weather events could result in short-term episodes resulting in some peak dry weather flows receiving only wet weather treatment (i.e., clarification and disinfection) and some wet weather flows by-passing treatment and disinfection. As such, maximum day or week flows will not accurately reflect the peak hourly amounts that receive only partial treatment, thereby resulting in higher than expected final effluent loads and concentrations. The hourly stress pattern based on the last six years of flow and quality records and appropriately scaled to 2031 and 2061 populations, was used to simulate the flows and concentrations the unit operations and treatment process are anticipated to experience and the corresponding effluent loads from the dry weather, wet weather, and by-passes. This simulation allows more accurate quantification of the predicted 24-hour composite effluent quality. The predicted effluent quality will be used to assess compliance with the performance criteria provided by the Program Team and identify possible changes to the unit operations, treatment process sizes and flow splits to improve compliance with established effluent requirements.

6.3.7 Flow Projections

Two independent approaches have been developed to estimate the projected peak flows to the SEWPCC for a 20-year design horizon corresponding to the year 2031, these are referenced to as the “collections systems approach” and a “plant-based approach”. The collection systems approach used in Section 4.2 estimates the flows conveyed to the SEWPCC via the sewer network system and approximates possible losses due to emergency overflows in response to excessive wet weather inflows. The plant-based approach considers only the flows received at the plant and inherently accounts for upstream behavior and losses. Both of the approaches assumed that the collection system delivery capacity to the SEWPCC will not change within the 20-year design period used for the SEWPCC upgrade and expansion. A comparison was performed on the projected flows to identify any major discrepancies that might exist in the projected flows. This review was intended to provide an independent quality check and assurance on the projected flows to improve the confidence in the flow projections while identifying any possible limitations or issues associated with the projections.

Figure 6.9 compares the projected unconstrained flows for both described approaches, and clearly illustrates that the vast majority of the flow projections are well matched and compare very favorably.

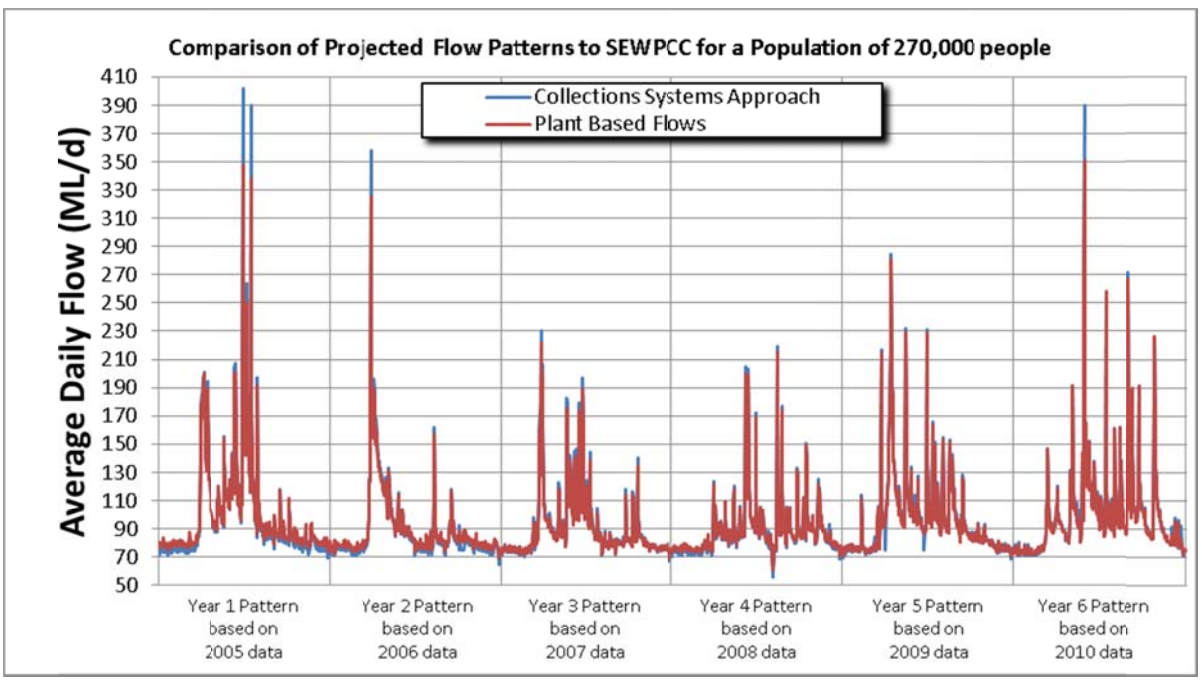


Figure 6.9: Comparison of Projected Flow Patterns to SEWPCC

The notable differences between the two approaches involve the 4 major wet weather induced flow events. The collection system approach predicts larger daily peak for these 4 events. In general, the collection system approach tends to predict slightly higher peak and low flows (in the order of 3 to 6%) than the plant based approach. The review confirmed that both approaches are well within the estimation confidence and accuracy bounds for such an exercise. Furthermore, the existing system hydraulic capacity of the interceptor connecting to the SEWPCC has an estimated peak hour capacity of 420 ML/d, and a peak day capacity of 325 ML/d. While it is possible to exceed the peak day hydraulic capacity during peak hour events, peak wet weather flows in the collection system need to be constrained and managed to reduce the risk of wide-spread basement flooding associated with elevated water levels in the collection system. This is a concern during conditions of excessive wet weather flows, especially during high river level conditions. As such, while peak day flows were projected to exceed 325 ML/d in both approaches, a limit of 325 ML/d was place on the peak day flows to constrain the flows to operating practices that would protect against wide-spread basement flooding. Specifically, under such extreme events flow shedding would be done at specific outfall locations in order to simultaneously reduce the hydraulic burden on the system and minimize the risk of wide-spread basement flooding.

Table 6.4 summarizes projected peak flows to the SEWPCC under constrained and unconstrained conditions for both approaches, and clearly demonstrates that the flow projections are highly matched and compare very favorably, numerically. Both approaches use the same dataset of historic flows (i.e., January 1, 2005 to December 2010, inclusive), and are scaled up to projected 2031 conditions based on the same population growth and same additional extraneous flows contributions from new developments. The key difference is that the collection system approach estimates unconstrained flows entering the collections on a system-wide basis and estimates the flows lost due to emergency overflows, while the plant based approach scales up flows as received at the plant, which inherently accounts for losses.

Table 6.4 - Comparison of SEWPCC Design Flows (ML/d) for a Projected Population of 270,000 People in the year 2031

Year = 2011 (current)	**Winter**		**Spring**		**Summer**		**Fall**	
Approach to Flow Projections	A	B	A	B	A	B	A	B
Average	48.3	48.3	70.7	70.7	71.8	71.8	59.4	59.4
Max of 30-Day Average	50.8	54.9	112	114	114	124	91.0	89.6
Max of 7 Day Average	56.0	56.0	159	159	168	168	114	114.0
Max Day	78.6	78.6	276	276	272	272	205	205
Year = 2031, unconstrained	**Winter**		**Spring**		**Summer**		**Fall**	
Approach to Flow Projections	A	B	A	B	A	B	A	B
Average	75.4	76.7	108	104	105	104	88.5	88.7
Max of 30-Day Average	79.8	84.3	160	155	174	167	124	125
Max of 7 Day Average	85.9	85.4	210	211	235	221	154	155
Max Day	114	111	390	351	402	348	272	267
Year = 2031, constrained	**Winter**		**Spring**		**Summer**		**Fall**	
Approach to Flow Projections	A	B	A	B	A	B	A	B
Average	75.4	76.7	108	104	105	104	88.5	88.7
Max of 30-Day Average	79.8	84.3	160	155	174	166	124	125
Max of 7 Day Average	85.9	85.4	**201**	**210**	**220**	**217**	154	154
Max Day	114	111	**300**	**324**	**324**	**324**	272	264

Notes : Approach "A" denotes a Collection System-based approach

Approach "B" denotes a Plant-based approach

** "Constrained" represents the actual hydraulic capacity of the existing interceptor conveyance system.*

A review of both methods identified a significant uncertainty on the reality of the projected peak wet weather flows for the design horizon of 2031, and the actual hydraulic capacity of the interceptor, especially under high river level conditions. Both approaches yielded virtually identical flow patterns and identified the same risks and uncertainties. For subsequent assessment purposes, the Plant Based approach will constrain maximum daily flows to a maximum of 325 ML/d to match collection system operating limits and offer protection against wide spread basement flooding. It is important to note, the Provincial Regulator has not imposed compliance limits on frequency or duration of overflows from the collection system. Should such requirements be imposed on the City of Winnipeg, the frequency, magnitude, and duration of peak wet weather flows may increase at the SEWPCC. Since both approaches yield essentially the same flow patterns and are both well within the accuracy required for this assessment, either approach is suitable for flow related decisions regarding plant hydraulics. The subsequent wastewater loading and compliance analyses are based on the **plant-based approach** and it is this approach that was adopted for application in compliance assessment purposes only. Flows and loads contained in Table 4.18 are the conditions to be used for design purposes.

6.3.8 Load Projections

A review of the available data was completed for the years 2005 to 2010, inclusive as presented Section 4. The approach taken in these analyses was to develop a peak factor relationship for each of the key compliance parameters and for each averaging period important for design (i.e., max: day; 7-day rolling average; and 30-day rolling average). The data was then categorized by season and then scaled up by population. To account for outliers the 98th percentile was used to select a load for design. It is critical to note that in these analyses flows and concentrations for the same day were used to derive the observed historic loads at the SEWPCC, and that peak loads did not necessarily occur at the peak flow. Assuming that the peak flows and loads occur at the same time is a common practice in design and tends to result in an additional margin of safety in design and is considered sufficient for initial sizing of unit operation and processes.

The stress pattern was based on the same historic data for flows and concentrations to develop a flow and loading pattern to assess the ability of the upgraded plant to treat the projected loads for various flow splits and configurations for the upgraded processes. The primary difference was that the actual flows and loads remained linked and scaled up, based on a population of 270,000 people, and was then routed through various alternative configurations and sizes to assess compliance with established effluent criteria. It is important to note that the data review was conducted on the historical data as part of the development of the stress pattern to identify obvious outliers and substantially remove them from the data set to create a better representation of the real flows and loads for compliance assessment reasons.

A comparison was completed on the projected loads to identify major discrepancies that might exist in the projected loads based on approach and methodology. This comparison was intended to provide an independent quality check and assurance on the projected loads to improve the confidence in the load projections, while also identifying any possible limitations or issues associated with the projections. The methodology used to estimate the loads associated with the Design Loading Analysis is provided in Section 4.3 and is based on a 98th percentile approach with a corresponding peak factor analysis referenced to annual averages. The approach used in the stress pattern analysis considers the exact same data set (2005 to 2010, inclusive) but also includes scaling up the actual historic loads in direct proportion to the population increase. Specifically, it does not attempt to develop peak factor relationships based on annual averages but rather directly scales up the loads based on population growth, while accounting for loads from Windsor Park sewer district on a year-round basis and discounting loads associated with HLW. Each load was estimated based on seasonal averages, 30 day rolling average, and 7 day average. The maximum value for each averaging period for each parameter was selected for comparison. The flow and loads were found to compare very favorably between methods, verifying the suitability of the data set for use in compliance assessment.

Table 6.5 compares the loads that were developed as part of the Design Loading Analyses (column "A") design loading analysis and the Development of the Stress Pattern (Column "B"),stress pattern development and clearly demonstrates that the load projections are highly matched and, on a numerical basis, compare very favorably (see Table 6.5).

Table 6.5 - Comparison of SEWPCC Design Loads (Kg/d) for a Projected Population of 270,000 People in the year 2031

Season	Period	TSS Loading		BOD5 Loading		TKN Loading		TP Loading	
	Approach	A	B	A	B	A	B	A	B
Spring	Average	21447	18049	21112	20726	4355	4046	633	619
	Max 30d rolling avg	24796	27606	22761	24997	4631	4778	661	703
	Max 7 d rolling avg	32610	34759	32423	31904	5345	5548	945	949
	Max Day	75506	71333	49575	48712	7266	6933	1227	1289
Summer	Average	18554	16371	19997	19807	3774	3674	614	562
	Max 30d rolling avg	22337	28324	22567	25087	3977	4676	608	690
	Max 7 d rolling avg	36414	36439	29069	27658	4827	4944	746	722
	Max Day	68964	65010	48494	48178	7403	6715	1511	1461
Fall	Average	16504	13828	19911	19652	3955	3677	588	567
	Max 30d rolling avg	17308	17455	20936	22656	4143	4253	612	638
	Max 7 d rolling avg	20956	22030	23180	23499	5201	4582	706	701
	Max Day	39666	38085	31856	32163	5808	6054	1029	1091
Winter	Average	15414	13059	20677	19780	3946	3663	592	567
	Max 30d rolling avg	18367	16364	22251	23483	4296	4227	637	636
	Max 7 d rolling avg	29793	28734	28671	30196	4810	4304	758	724
	Max Day	38141	36941	32043	33396	6306	5925	1064	1152
Annual Average		**16200**	**15361**	**19440**	**19994**	**3780**	**3766**	**567**	**579**

Notes : Approach "A" denotes the approach used in the Design Loading Analyses
Approach "B" denotes the approach used in the Stress Pattern Development

It is important to note that the reporting of the 30-day rolling average reported the end of the average period, that is, show the 30 day rolling average result on day 30.

6.3.9 Performance Capacities of Unit Operations and Processes

The following performances shown in Table 6.6 were considered to be representative of the parameters being assessed.

Table 6.6 – Performance Values for Various Processes

	Removal Efficiencies				Expected Effluent Quality (mg/L)				
Component	TSS	BOD	TKN	TP	TSS	BOD	Ammonia	TN	TP
CEPT	50%	35%	15%	75%	*Variable*	*Variable*	*Variable*	*Variable*	*Variable*
BAF	***Fixed***	90%	***Fixed***	***Fixed***	12.0	*Variable*	4 or 1.5	12.0	0.8
HRC	85%	60%	40%	80%	*Variable*	*Variable*	*Variable*	*Variable*	*Variable*
Bypass	0%	0%	0%	0%	*Variable*	*Variable*	*Variable*	*Variable*	*Variable*

Based on a literature review and expectations related to the type of clarifiers employed at the SEWPCC, upgrading and improving the performance of existing primary clarifiers by the use of chemicals to a CEPT, including the discontinuation of co-thickening with WAS will at a minimum achieve the efficiencies noted in Table 6.6. The expected performance of CEPT was assumed to provide an influent loading to the BAF that was within its treatment capacity. It is understood that BAF treatment is strongly affected by TSS concentrations and as the influent TSS is reduced the better the performance of the BAF to reduce soluble carbon and nitrogen constituents. It is strongly recommended that a stress test be performed on at least one of the existing clarification units to confirm its actual performance to more accurately determine its performance capacity in CEPT mode, and the flow and load that can be safely put through the clarifier to achieve the effluent quality required for the BAF to perform within its design limits.

Based on a limited literature review, information provided by Suppliers, and experience elsewhere, HRC through physical processes and chemical additions should be able to achieve the efficiencies noted in Table 6.6. The expected performance of HRC was assumed to provide an effluent quality that would be within the treatment requirements for BAF and/or for disinfection based on a chlorination and de-chlorination system. Since cost assessments are not part of this assignment, analyses have not occurred to determine the most cost-effective split between the HRC and the upgrading and expansion of the existing primary clarifiers to CEPT. Nonetheless, based on discussions with the Program Team it has been assumed that it is more cost effective to add HRC rather than expand the existing clarifiers, resulting in an upgrade of the exiting primary clarifiers to CEPT and any additional capacity required would realized through the addition of new HRC units.

Peak hourly flows greater than 325 ML/d receive only screening and grit removal. The diluted concentrations and associated loads are blended back in with the treated flows.

Preliminary calculations where performed using the loads and flows as summarized in Table 4.18 to determine the effluent quality required from the BAF in concert with the HRC in order to achieve compliance with final effluent criteria as listed in Table 6.1. The calculations revealed that to achieve the rolling monthly averages for Total Nitrogen (TN) and Total Phosphorus (TP), the required Total Suspended Solids (TSS) would need to be about 12 mg/L. Specifically, a portion of the TSS will contain nitrogen and phosphorus, and will need to be accounted for in the estimation of final effluent TN and TP. Based on cellular composition of bacteria commonly found in wastewater treatment, the cell mass is composed of: 45 to 55% carbon (50% average)[2]; 12 to 16% of nitrogen (14% average)[2]; and 2 to 5% phosphorus (3% average)[2]. At 12 mg/L of TSS, the effluent will contain elemental concentrations: carbon of 6 mg/L; nitrogen of 1.7 mg/L; and phosphorus of 0.36 mg/L. Since the final effluent compliance requirements are less than or equal to a specific value, it is a common practice to set a performance about 80% of the compliance requirement to provide a margin of safety to allow for minor process variations. As such, the 30-day rolling averages for TN and TP reduce from 15 mg/L- N to 12 mg/L- N and from 1.0 mg/L-P to 0.8 mg/L – P, respectively. Based on these relationships, the maximum allowable soluble nitrogen (i.e., nitrite + nitrate + ammonia) should not exceed 10.3 mg/L and the maximum allowable soluble phosphorus (i.e., ortho-P) should not exceed 0.44 mg/L. It is also important to note that reducing soluble phosphorus to levels below 0.5 mg/L by the use of metal salts requires an increasing amount of metal salts due to the dilute concentration of the wastewater. Removal of too much soluble phosphorus by CEPT could also limit the soluble phosphorus required for bacterial synthesis during nitrification and denitrifying stages of the BAF process.

Based on the month-specific effluent ammonia criteria that are not-to-exceed on a daily basis, the greatest compliance challenge will be experienced, especially in the summer months. Based on the maximum allowable ammonia effluent load of 607 kg/d - N for the month of August, the corresponding maximum allowable concentrations have been calculated for projected flows in this sensitive period, as summarized in Table 6.7 below.

Table 6.7 – Maximum Allowable Effluent Ammonia Concentrations in August for Specific Flows

Averaging Period	Projected 2031 Summer Flows (ML/d)	Maximum Allowable Ammonia Concentration (mg/L as N)
Average Day	105	5.8
Maximum of 30-Day Average	174	3.5
Maximum of 7-Day Average	220	2.8
Maximum Day	324	1.9

As shown in Table 6.7, the final effluent concentrations that need to be achieved decrease with increasing flows for the month of August to remain within the maximum allowable daily ammonia load limit. Achieving compliance for projected 2031 summer flows becomes very challenging and potentially very costly to build a facility that can maintain compliance during critical periods with high flows associated with peak day and peak week. As such, the size and performance of the BAF is a key aspect of achieving compliance with the ammonia effluent requirements. Since negligible ammonia is removed by CEPT and HRC, the primary removal mechanism for ammonia is BAF treatment. As such, the ability of BAF to achieve low ammonia concentrations

[2] Wastewater Bacteria, M.H. Gerardi, 2006

in the summer months will be a critical factor in the design and operation of the nitrogen removal facility. The initial design of the BAF was based on a simultaneous Nitrification and De-Nitrification (NDN) operation. To achieve the low levels of ammonia that are required may dictate the BAF to be operated in separate stages for Nitrification, and external carbon-induced De-Nitrification, these needs will be a critical consideration in subsequent engineering design phases. Based on discussions with BAF suppliers a NDN BAF facility can be designed to achieve an effluent ammonia concentration of 4 mg/L under maximum flows, loads, and minimum summer operating temperature, while a two-stage BAF can be designed to achieve an effluent ammonia concentration of 1.5 mg/L under maximum flows, loads, and minimum summer operating temperatures.

It was assumed that disinfection of pathogens, and chemical removal of phosphorus could be achieved by adjusting the dosage as required and therefore not simulated through the stress pattern analyses. The sludge generated from the unit processes and operations for spring maximum month was assessed under a different task. The primary purpose of this exercise was to simulate the resulting effluent quality based on removal efficiencies of the unit process for various flow splits.

A preliminary mass balance analysis was conducted as part of another task associated with the Biosolids Implementation Strategy. All residual solids from CEPT, BAF, and HRC will be sent to a sludge thickening and blend tank before they are digested and dewatered. The reject liquid stream from the blend tank is referred to as filtrate. The resulting flow is expected to be about 0.6 ML/d on a peak monthly basis for projected 2031 flows and loads. The inclusion of digestion and dewatering at the SEWPCC will result in a highly concentrated reject liquid stream referred to as centrate. The centrate stream typically has a low carbon content but very high concentrations of phosphorus and ammonia and can add a significant load to the main stream if not treated prior to its introduction to the main stream. The average centrate flow is expected to be about 0.6 ML/d for the same 2031 peak month flows and loads. As well, depending on the dewatering process, centrate may contain a high TSS load, and would require clarification to minimize its impacts on main stream processes. It is not possible to confidently estimate centrate loads at this time because they are very dependent on the processes selected for solids treatment. There are many variations of digestion and dewatering systems, each option correspondingly produces difference volumes and strengths of centrate. The mass balance analysis done for biosolids handling and treatment was to estimate the approximate sizing of units required for solids treatment based on digestion and dewatering. Nonetheless, a review was completed on the NEWPCC centrate characteristics and this was considered representative of the centrate concentrations at the SEWPCC if digestion and dewatering was implemented.

A review of the resulting centrate load found that it could present a significant recycle load to the BAF. Based on the Centrate treatment processes used at the NEWPCC, it was assumed that a similar system could be employed and effectively reduce the nitrogen, phosphorus and TSS loads such that they would represent a minor contribution to the overall influent load on the main stream processes and therefore can be ignored in this exercise. Table 6.8 summarizes the influent and effluent concentrates associated with the NEWPCC centrate treatment process. It is important to note that these are average concentrations and that fluctuations in the performance and effluent quality will need further assessment as part of the schematic design. The treated filtrate and centrate return flows have at least three potential locations where they can be introduced into the main stream process: 1) the head end of the plant (recommended at this time); 2) immediately upstream of CEPT; and 3) into the intermediate pumping station feeding the BAF, and will assessed as part of subsequent engineering design phases.

Table 6.8 – NEWPCC Average Centrate Influent and Effluent Concentrations

Parameter	TKN mg/L	NO3 mg/L	TN mg/L	NH4 mg/L	Psol mg/L	TP mg/L	tCOD mg/L	sCOD mg/L	TSS mg/L	Alk mg/L
Average influent concentration	800.0	46.0	846.0	700.0	17.4	53.0	600.0	340.0	229.0	2000.0
Average effluent concentration	22.0	7.0	29.0	10.0	9.5	19.0	500.0	240.0	88.0	500.00
Removal efficiency	**97%**	85%	**97%**	**99%**	**46%**	**64%**	17%	29%	62%	75%

6.3.10 Compliance Assessment

In the development of the stress pattern, a conservative approach was taken to populate missing data in the database with values that were within the upper limits of the monitored range of values (e.g., one standard deviation above the linear regression of the available data). The data was then reviewed to ensure that specific relationships between related parameters were within expected ranges.

Based on the forgoing, the stress pattern was applied for the following scenarios:

Scenario 1

- All flows up to 420 ML/d receive screening and grit removal
- Removal efficiencies as noted in Table 6.6
- Expected BAF effluent quality as noted in Table 6.6
  - Use 4 mg/L maximum effluent ammonia concentration from BAF in NDN mode
- Flows up to 200 ML/d through CEPT and BAF, with UV disinfection
- Flows greater than 200 ML/d up to 325 ML/d through CEPT and HRC, followed by chlorination and dechlorination
- Peak hourly flows greater than 325 ML/d by-passed around clarification and blended

Scenario 2

- Same as scenario 1 except BAF operated in separate nitrification and de-nitrification modes to achieve a maximum effluent ammonia concentration of 1.5 mg/L

Scenario 3

- Same as scenario 2 except:
  - flow to BAF reduced to 180 ML/d
  - flow to HRC increased to 145 ML/d

Scenario 4

- Same as scenario 3 except:
  - flow to BAF reduced to 170 ML/d
  - flow to HRC increased to 155 ML/d

Scenario 5

- Same as scenario 4 except:
  - flow to BAF reduced to 160 ML/d
  - flow to HRC increased to 165 ML/d

A summary of the model outputs for the maximum conditions simulated for the 5 scenarios are summarized below in Table 6.9 and 6.10, and graphically illustrated for Scenarios 1 and 2 in Figures 6.10 and 6.11, respectively.

Table 6.9 – Maximum Predicted Values for Various Treatment Scenarios for 2031

Scenario	BAF +HRT (ML/d)	TSS 30d Ave (mg/L)	TN 30d Ave (mg/L)	TP 30d Ave (mg/L)	BOD 30d Ave (mg/L)	Ammonia Non-Compliance with Month Specific Daily Never-To-Exceed Limits
1	200[a] +125	12.8	12.1	0.80	19.7	13 events in 6 years
2	200[b] +125	12.8	12.1	0.80	19.7	2 events in 6 years
3	180[b] +145	13.1	12.2	0.80	19.7	5 events in 6 years
4	170[b] +155	13.3	12.3	0.80	19.7	7 events in 6 years
5	160[b] +165	13.5	12.4	0.80	19.7	11 events in 6 years
Notes: a denotes BAF ammonia effluent of 4 mg/L b denoted BAF ammonia effluent of 1.5 mg/L						

Table 6.10 – Estimated Year-Specific Ammonia Non-compliances based on the Stress Pattern Analyses for 2031

Scenario	Year 1 based on 2005 data	Year 2 based on 2006 data	Year 3 based on 2007 data	Year 4 based on 2008 data	Year 5 based on 2009 data	Year 6 based on 2010 data
1	1	1	0	2	1	8
2	0	0	0	1	0	1
3	1	1	0	1	0	2
4	1	1	0	1	0	4
5	1	1	0	2	1	6

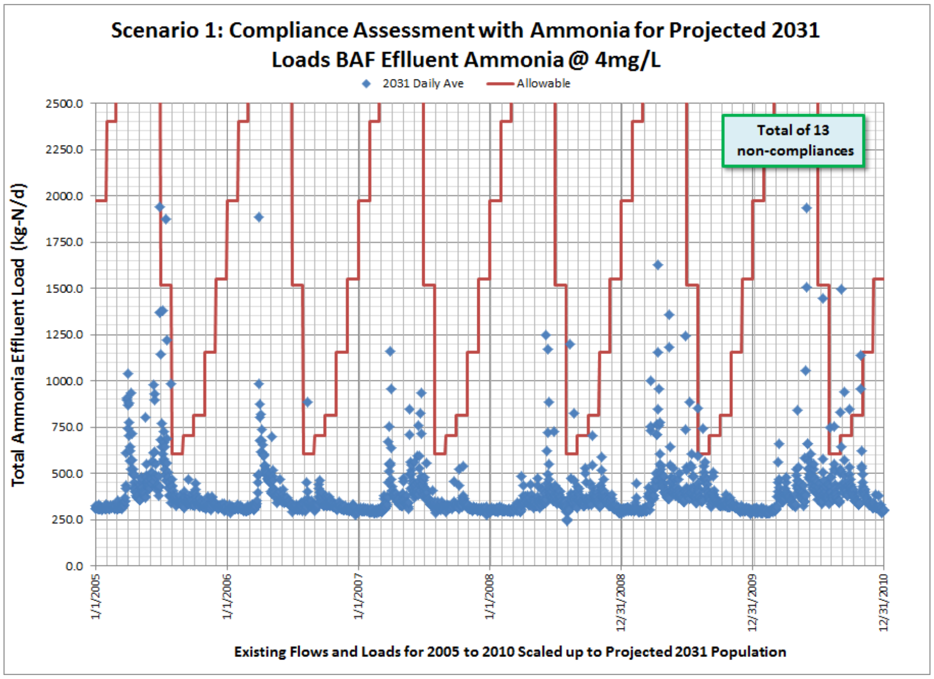


Figure 6.10: Estimated Ammonia Non-Compliances based on the Stress Pattern Analyses for Scenario 1

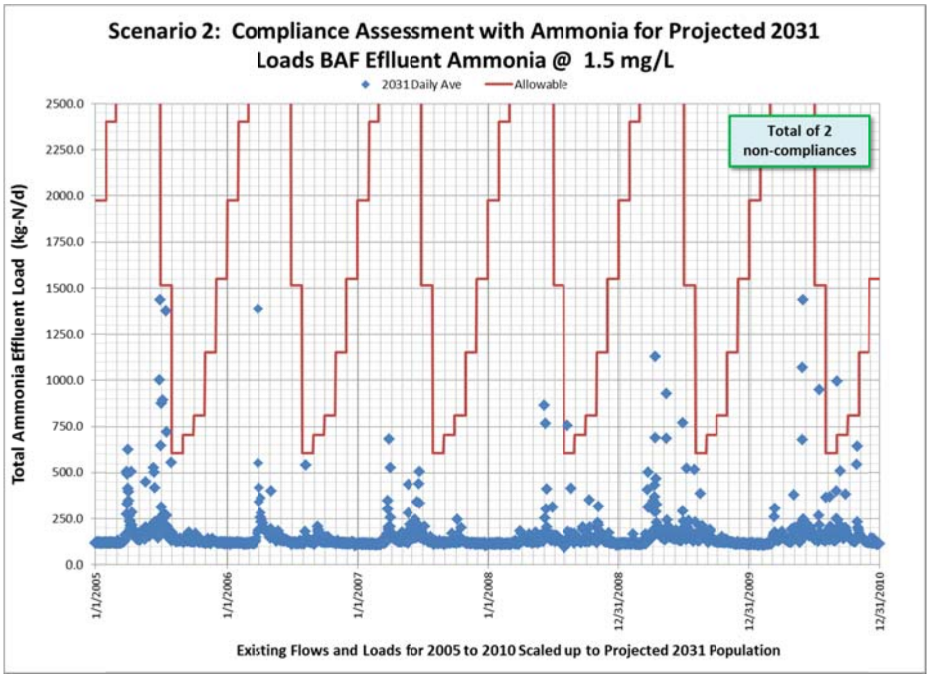


Figure 6.11: Estimated Ammonia Non-Compliances based on the Stress Pattern Analyses for Scenario 2

6.3.11 Summary and Discussion

The results of the simulations clearly indicate that compliance with the ammonia effluent criteria is the controlling factor on the operational requirements for the SEWPCC upgrade and expansion. The size, performance, and operational requirement of the BAF is the key unit process influencing compliance with the ammonia effluent criteria. It is unlikely that full compliance for ammonia can be achieved without significantly over-sizing the BAF treatment process based on the scenario of projected flows and loads associated with a design population of 270,000 people in the year 2031. This fact will require a deliberation by the Program Team to determine if there is an acceptable trade-off in BAF size (and ultimately cost) and compliance with the effluent ammonia criteria. Based on representative removal efficiencies and expected performance of the BAF, full compliance for TSS, $cBOD_5$, TN and TP with the criteria established by the Program Team is possible for a well operated plant under normal conditions.

Since ammonia is a governing parameter in the design of the BAF treatment process, it is recommended that a more rigorous assessment be done as part of the schematic design for performance assessment and compliance reasons. Nonetheless, the simulations that were done using the stress pattern indicated that initial sizing of the unit processes and operations are within acceptable bounds for subsequent engineering design purposes. The results also suggest that optimization in subsequent engineering design phases might be able to reduce the size of the BAF depending on the level of risk the Program Team is willing to accept in terms of reporting minor and short-term excursions associated with the effluent ammonia criteria.

The modeling of the unit processes and operations as part of this report is based on performance capabilities provided by suppliers that can be readily achieved for the technologies being considered. As engineering proceeds and more detailed information becomes available, it might be possible to further refine and optimize the unit processes and operations and reduce the size of certain components. At this stage, based on the performance capabilities provided by suppliers for the BAF and HRC, in conjunction with typical removal efficiencies for the existing clarifiers, the assessment validates that the proposed technology has the capacity to achieve compliance with the criteria as provided by the Program Team It is important to note that in wet years there is the potential for minor and rare excursions of the ammonia criteria.

7.0 Conceptual Development

This section describes the major process components identified for conceptual design and provides a conceptual layout that defines their interrelationship. It must be recognized that at this early stage in the design process there are many details that have not yet been developed and the components and layout have not been optimized. Optimization of the components and layout will take place during subsequent design phase revisions.

In this section, the major process components are defined in terms of critical core capabilities. In some cases, specific suppliers are mentioned, not because these are the recommended suppliers, but to demonstrate that the proposed process components and layout can be designed to operate as shown.

7.1 COLD WEATHER REQUIREMENTS

This section discusses the cold weather operational considerations for the proposed BAF facility. Most of the current infrastructure at the SEWPCC is either indoors (e.g., primary clarifiers, secondary clarifiers etc.) or are covered (secondary high purity oxygen bioreactors).

This section of the report was completed based on literature search and discussions held with John Meunier Inc., Montreal (BIOSTYR process) and Degremont Technologies (BIOFOR process), the two major BAF suppliers in the market place today.

7.1.1 Cold Weather Issues

A majority of the BAF plants in Canada are open tank design. These include plants at Canmore, Alberta (BIOFOR); Thunder Bay WWTP, Ontario (BIOFOR), Windsor WWTP, Ontario (BIOFOR), Chateauguay, Quebec (BIOSTYR) and most recently the Ravensview WPCF in Kingston, Ontario (BIOSTYR). The only covered plant BAF facility in operation in Canada is located in Sherbrooke, Quebec. The 80 ML/d facility is located in a building, and each filter cells are covered by individual fibreglass domes to protect the building interior from corrosion. Degremont indicated that they have installations in Norway which are located inside a building due to extreme cold. In the United States, the Metropolitan Syracuse WWTP, NY operates a tertiary BAF that is completely inside a building. In Europe, Geneva, Switzerland is an example of an enclosed BAF (BIOSTYR) operating in a cold climate.

In discussions with the two Suppliers and literature search, the following key operational considerations were shortlisted:

- *Operator safety and comfort*: Due to extreme cold weather periods in Winnipeg and potential for icing in the walkways for open structures, there is certainly a concern for day to day operational aspects of the process during the winter months. However, the BAF process can be highly automated requiring less operator input except for a quick visual check of the filter cells.

- *Freezing potential*: Based on feedback received from the suppliers and their experiences with ongoing plants in Canada, freezing potential is not an issue and poses no operational problems with open tanks

- *Algae control*: Both of the BAF processes i.e., BIOSTYR and BIOFOR are based on upflow configuration and there is open water at the top of the tank. Some plants with open tankage have therefore experienced algae problems. To address this concern, John Meunier Inc. has installed nets over the filter cells (e.g., Ravensview WPCF in Kingston, Ontario). The effectiveness of this approach is unknown at this time. Having the tanks covered will certainly address the algae concern in a plant.

- *Media access and replacement*: Access to BAF media is a key operational and maintenance requirement in such plants. Typical media loss is about 3% annually. In addition, height of the media needs to be checked from time to time. An open tank design provides the maximum flexibility to accessing the media. However, any inspection would have to be completed in late fall prior to the winter months. A covered design will provide the maximum flexibility is this case.

- *Odor mitigation*: There has been reported problems with odors with BAF plants, particularly, utilizing a down-flow type design. Both the systems being considered are up-flow type BAF systems and only the treated effluent collected on top of the filter is exposed to the atmosphere.

7.1.2 Recommendation

Considering Winnipeg's climate of extreme cold to hot conditions and the potential for algae growth in the open water, Stantec recommends a light building to be constructed on top of the BAF structure. The building does not necessarily have to be completely enclosed, but will provide protection from the weather elements discussed in this section. Further details can be developed in the Preliminary Design. Some of the examples are shown in Figures 7.1 to 7.3.

Figure 7.1: Partially Covered Building over BAF tanks (Source: VMS, Paris)

Figure 7.2: Covered Building in a BAF plant at Boisbriand, Quebec (Source: John Meunier)

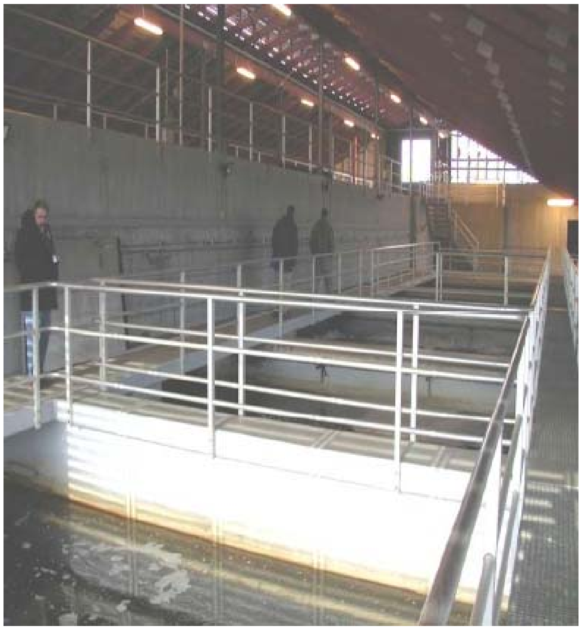


Figure 7.3: BAF Plant in Norway (Source: Slitt and Welch, 2005)

7.2 HEADWORKS

The required upgrades to the existing headworks facility to accommodate the increased flows and proposed process design are detailed in the subsections below:

7.2.1 Raw Sewage Pumps

The existing raw sewage pumping system has a rated capacity of 364 ML/d. As the wet well level increases, the pumping capacity increases, and the highest observed flow from the raw sewage pumping system is 388 ML/d. The Conceptual Design Report (Stantec, 2009) identified that the total pumping capacity could be increased to 422 ML/d by replacing one of the smaller pumps with a larger pump. It is proposed to replace G102-RSP with a pump rated at 114 MLD to accommodate the required pumping capacity of 420 MLD.

7.2.2 Screens

The existing screening system includes three (3) climber type 12 mm bar screens installed in 1.829 mm wide channels. Each screen has a hydraulic capacity of 180 ML/d for a total screening capacity of 540 ML/d.

The proposed Biological Activated Filter (BAF) process requires 6 mm perforated plate screening at a minimum. Three (3) new 6 mm perforated plate screens are proposed to be installed in the existing screen channels. The rated capacity of each screen will be 140 ML/d for a total screening capacity of 420 ML/d. The proposed design was validated based on the information supplied by Huber Technology and the design criteria are summarized in Table 7.1. Evaluation of alternative perforated plate screens is required in subsequent design phases in order to select the most appropriate screen for the SEWPCC. Refer to Figure 7.4 for the proposed layout of the screen channels.

Table 7.1 – Screening Design Criteria

Design Element	Criteria-Specified Value
Peak Hydraulic Capacity	420 ML/d (140 ML/d / screen)
Number of Screens	3
Screen Clear Opening	6 mm
Channel Width	1.55 m
Screen Width	1.78 m
Screen Downstream Water Level	1.983 m
Screen Headloss at 10% Blinding	200 mm
Screen Headloss at 50% Blinding at 0.6 m/s	348 mm
Screen Upstream Water Level	2.331 m

To determine the feasibility of retrofitting the existing channels with perforated plate screens it has been assumed that the maximum water level cannot exceed the top of the raw sewage discharge pipes. It has been assumed that the top of the raw sewage discharge pipes is 2.398 meters above the screen channel floor. This elevation is based on a single measurement of one discharge pipe. Further measurements to confirm the elevation of all discharge pipes is required in subsequent design phases.

7.2.3 Grit Removal

The existing grit removal system includes two (2) 9.1 m square aerated grit tanks. The grit tanks are aerated by constant speed duty / standby blowers and settled grit is removed by constant speed duty / standby grit pumps. Settled grit is pumped to two (2) grit classifiers and the dewatered grit is discharged to the grit / screenings bin. At a hydraulic retention time (HRT) of 4 minutes the hydraulic capacity of the existing grit system is 264 ML/d.

The proposed BAF and HRC processes both require grit removal. The untreated bypass to the river also requires screening and grit removal. It is proposed to split the flow following screening into the main process stream and the side stream.

The existing grit system will be retained and de-rated from 264 to 200 ML/d (i.e., HRT = 5.4 minutes). De-rating the capacity of the existing grit system to 200 MLD will improve performance (refer to page 7.19 of the June 2009, SEWPCC Upgrade/Expansion Conceptual Design Report prepared by Stantec,) and simplify flow splitting between the side stream and main stream. Flow up to 200 ML/d will pass through the existing grit system and flow by gravity to the existing primary clarifiers. Flow up to 150 MLD will pass through the existing primary clarifiers and then be pumped to the BAF, while the remaining 50 MLD will be diverted to the sidestream.

Flow in excess of 200 ML/d and up to the plant capacity of 420 ML/d will overflow a newly constructed weir to a proposed high rate vortex grit removal system. Effluent from the high rate vortex grit removal system will be further split with up to 125 ML/d flowing by gravity to the side stream HRC process and the additional 95 ML/d bypassing further treatment. Settled grit will be pumped to a new grit dewatering and classification system.

The proposed vortex grit tanks will be located east of the existing grit tanks. This location blocks the south truck access to the existing screenings and grit bin. It is proposed to expand the screen room to the east, rotate the existing screenings and grit bin and add a new overhead door to provide access to the bin. Refer to Figure 7.4 for a layout of the proposed screenings and grit facility.

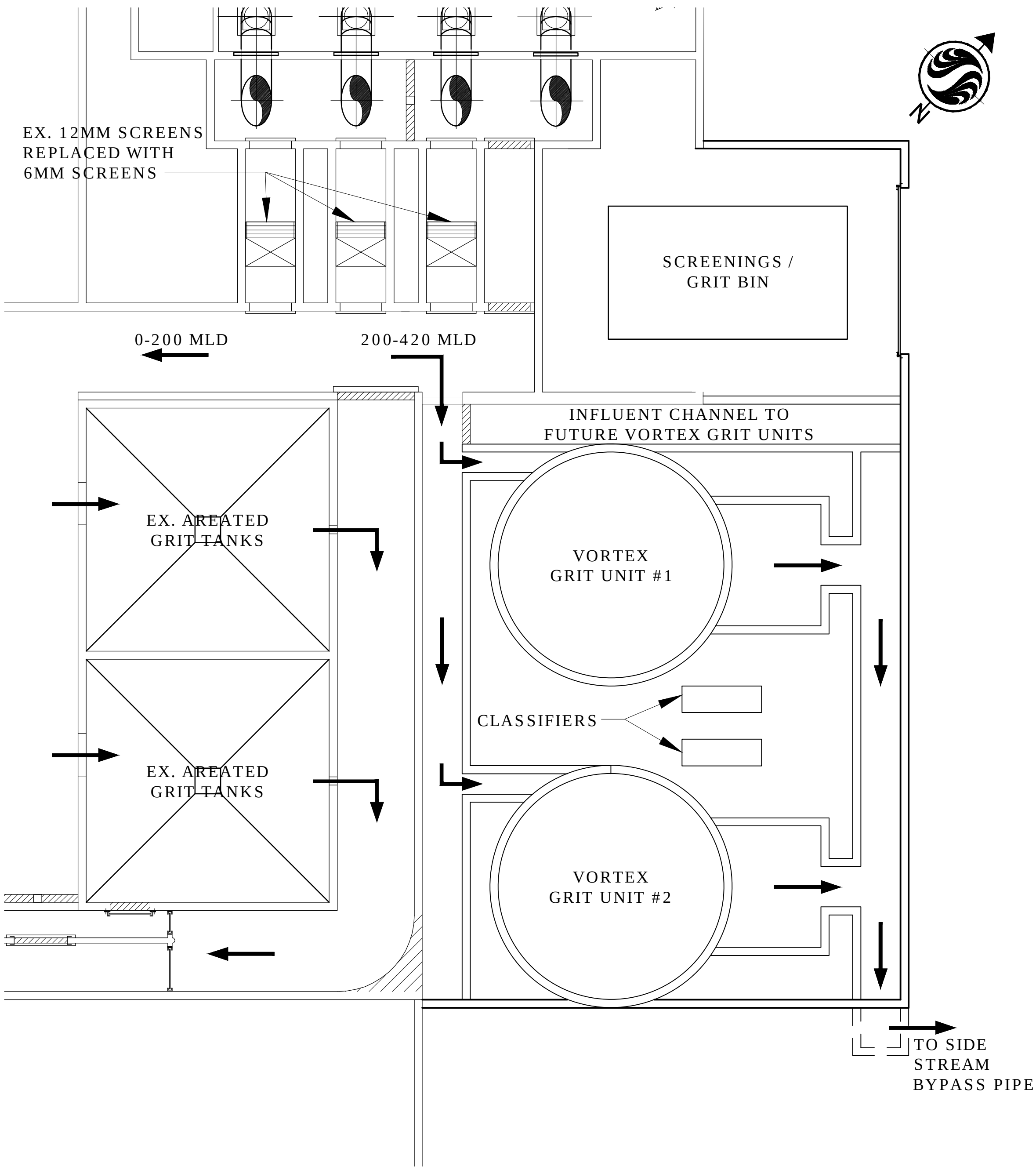
The design of the high rate vortex grit removal system is based on the Storm King® units supplied by Hydro International and the design criteria are summarized in Table 7.2.

Table 7.2 – Grit Removal Design Criteria

Description	Value
Peak Hydraulic Capacity	220 ML/d
Number of tanks	2
Tank Diameter	8.5 m
Headloss at Peak Flow	230 mm
Girt Removal	95% of 150 micron or larger

The main floor of the proposed vortex grit system will match the main floor in the existing grit room. The grit classifiers will be located at this level between the grit tanks. The separated grit will be conveyed to the existing screening and grit bin in the screening room. The grit pumps will be located on a lower level that is accessed from existing Gallery No. 2. The invert of the proposed vortex grit tanks will be located approximately 5.4 m below the existing grade, while the water level in the tanks will be approximately 2.45 m above the existing grade.

To facilitate maintenance on the existing aerated grit system, the design will provide a provision for the proposed vortex grit system to be used instead of the aerated grit chambers during dry weather flow. A pipe that allows for flow transfer between the mainstream and side stream processes downstream of grit and upstream of primary clarification is required. The pipe will allow for flow to be directed from the side stream bypass pipes to the existing primary clarifiers during periods when the aerated grit chambers are out of service for maintenance during dry weather flows. During wet weather flow, the pipe will transfer flow in excess of 150 ML/d, but less than 200 ML/d from upstream of the primary clarifiers to the proposed high rate clarifiers via the side stream bypass pipe.



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HOR. SCALE: VERTICAL:	1:250	RELEASED FOR CONSTRUCTION:	
DATE	2011.11.01	DATE	

ENGINEER'S SEAL

TENDER NO.

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

HEADWORKS CHANNEL CONCEPTS

CITY DRAWING NUMBER
FIGURE 7.4

SHEET OF

7.2.4 Headwork Channel Modifications

The existing headworks channels are oversized for dry weather flows and as such experience significant grit settlement. As noted in the Conceptual Design Report (Stantec, 2009), channel modifications are required to mitigate against grit settlement. The channel modifications will be further investigated in the preliminary design phase.

7.3 CHEMICALLY ENHANCED PRIMARY TREATMENT

The required upgrades to the existing primary clarification facility to accommodate the increased flows and proposed process design is as follows:

7.3.1 Primary Clarification

Presently, there are three (3) Primary Settling Tanks (PST) at the SEWPCC facility. Primary Settling Tank No. 1 (PST-1) and 2 (PST-2) each have a peak hydraulic capacity of 42 ML/d each, while Primary Settling Tank No. 3 (PST-3) has a peak hydraulic capacity of 90 ML/d; for a total peak hydraulic capacity of 174 ML/d. The current peak surface overflow rate in co-thickening mode [primary sludge is co-thickened with Waste Activated Sludge (WAS)] is calculated as 89.7 $m^3/m^2/d$ based on a total existing surface area of 1939 m^2 for the three PSTs.

For the proposed upgrade and as per the recommendations of the Process Selection Report (Veolia, 2011), the existing PST will be operated as a CEPT process upstream of the proposed BAF process. The main drivers for the CEPT process is enhanced removal of TSS and particulate BOD_5. Significant removal of TP will also be implemented via chemical addition (coagulant and polymer) to maintain the target TP level in the final effluent. A concept of the proposed CEPT is shown in Figure 7.5.

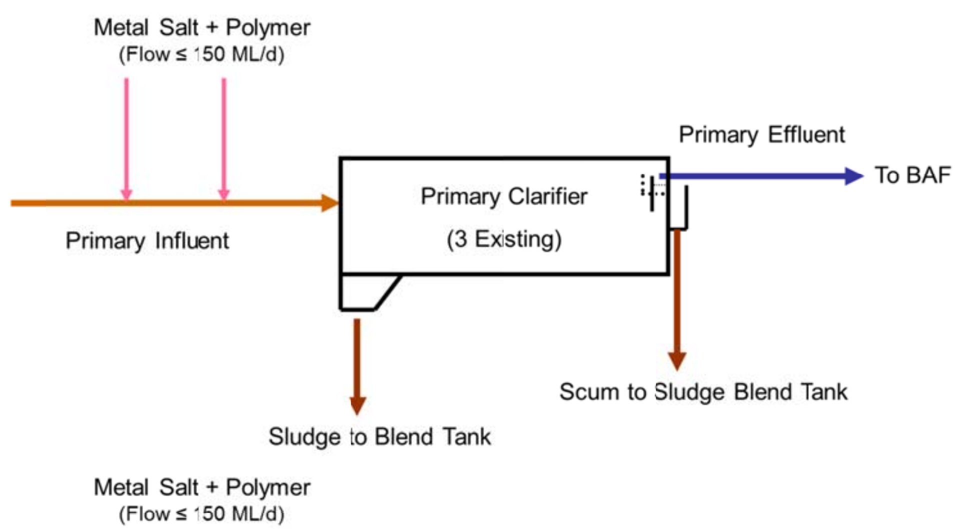


Figure 7.5: Proposed CEPT Schematic

7.3.2 Flow Distribution and Calculation of Surface Overflow Rate (SOR)

Currently, screened and degritted sewage flows through a 3.0 m-wide open channel from the grit removal system to the primary clarifier distribution channels. The distribution channels include 2 – 3.0 m-wide open channels, with one channel serving PST Nos. 1 and 2 and the second serving PST-3. The distribution channels are fitted with stop logs and baffle walls for uniform flow distribution and for solids splitting between the clarifiers. The stop logs can be used to take any portion of the channel out of service for maintenance purposes. A storm/emergency bypass channel, located along the length of PST-2 and PST-3, allows a portion or all of the flows to bypass primary treatment.

For the purpose of the project definition phase of the work, it was assumed that all flows up to 150 ML/d (peak hour basis) will be handled through the CEPT process. While the existing clarifiers are believed to be capable of treating a flow greater than 150 ML/d in CEPT mode, it is prudent at this point in design to down-grade the capacity of the existing primary clarifiers so that there is a high degree of confidence that primary effluent TSS will not overload the BAF and degrade its treatment performance. It is recommended that stress testing be performed on one of the existing clarifiers to more accurately assess its treatment capacity and ability to handle a higher SOR and the correspondingly higher flow. Flows in excess of 150 ML/d will be diverted to a new side-stream HRC up to 325 ML/d. Peak hourly flows in excess of 325 ML/d would be bypassed. As such, all flows up to the projected maximum day flow of 325 ML/d will receive a combination of CEPT and HRC followed by disinfection prior to being discharged to the river.

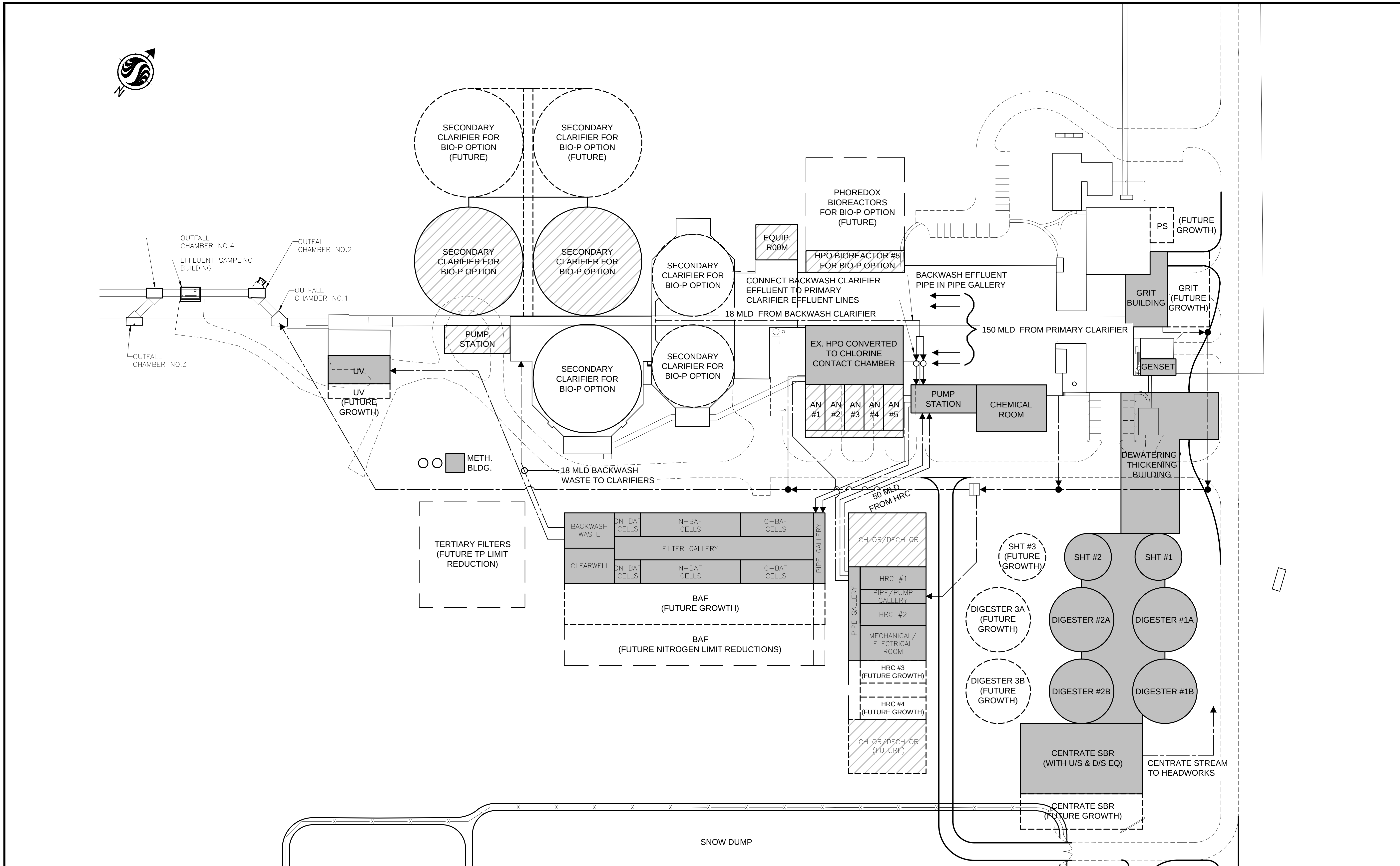
Based on these proposed flows, plant design criteria and existing surface area of 1939 m^2, the surface overflow rates (SOR) can be summarized as follows:

- SOR at Average Annual Flow of 94.5 ML/d = 48.7 $m^3/m^2/d$ or 2.03 m/h
- SOR at Max Hourly Flow of 150 ML/d = 77.4 $m^3/m^2/d$ or 3.2 m/h

As applicable to conventional primary clarification, the system design of CEPT is still governed by the surface overflow rate (SOR). Published value of SOR for CEPT ranges from 2.0 ~ 3.0 m/h under annual average conditions and from 4 to 5 m/h under peak hourly flows. As stated above, following stress testing, the maximum flow capacity of the existing PSTs will be established in CEPT mode.

7.3.3 Estimated Performance of CEPT

The performance estimates or removal efficiencies of the CEPT process for the key parameters were based on a review of the existing PSTs and literature on similar CEPT systems. It should be noted that the existing PST will no longer receive WAS for co-thickening in the proposed upgrade. This by itself will allow the PST to operate with the same level of efficiency with slightly higher SOR. In addition, the use of chemical enhancement will allow the PST to provide a much higher removal efficiency of TSS and BOD_5 along with TP and particulate TKN. The extent of soluble BOD_5 and soluble TKN removal will impact the overall removal efficiencies for these parameters. A summary of removal efficiencies for various parameters is summarized in



LEGEND

(shaded)	2031 (20 YR)
(hatched)	NEW CONSTRUCTION FOR BIO-P OPTION
— — —	2061 (50 YR)
— – —	TP & TN LIMITS
SHT	SLUDGE HOLDING TANK
HRC	HIGH RATE CLARIFICATION
BAF	BIOLOGICALLY AERATED FILTERS

1:750 0 7.5 22.5 37.5m

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. ELEV.

NO.	REVISIONS	DATE	BY

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DATE	2011.11.30	DATE	

ENGINEER'S SEAL

TENDER NO.

Winnipeg
THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

SITE PLAN - 2031 FLOWS

CITY DRAWING NUMBER FIGURE 7.6

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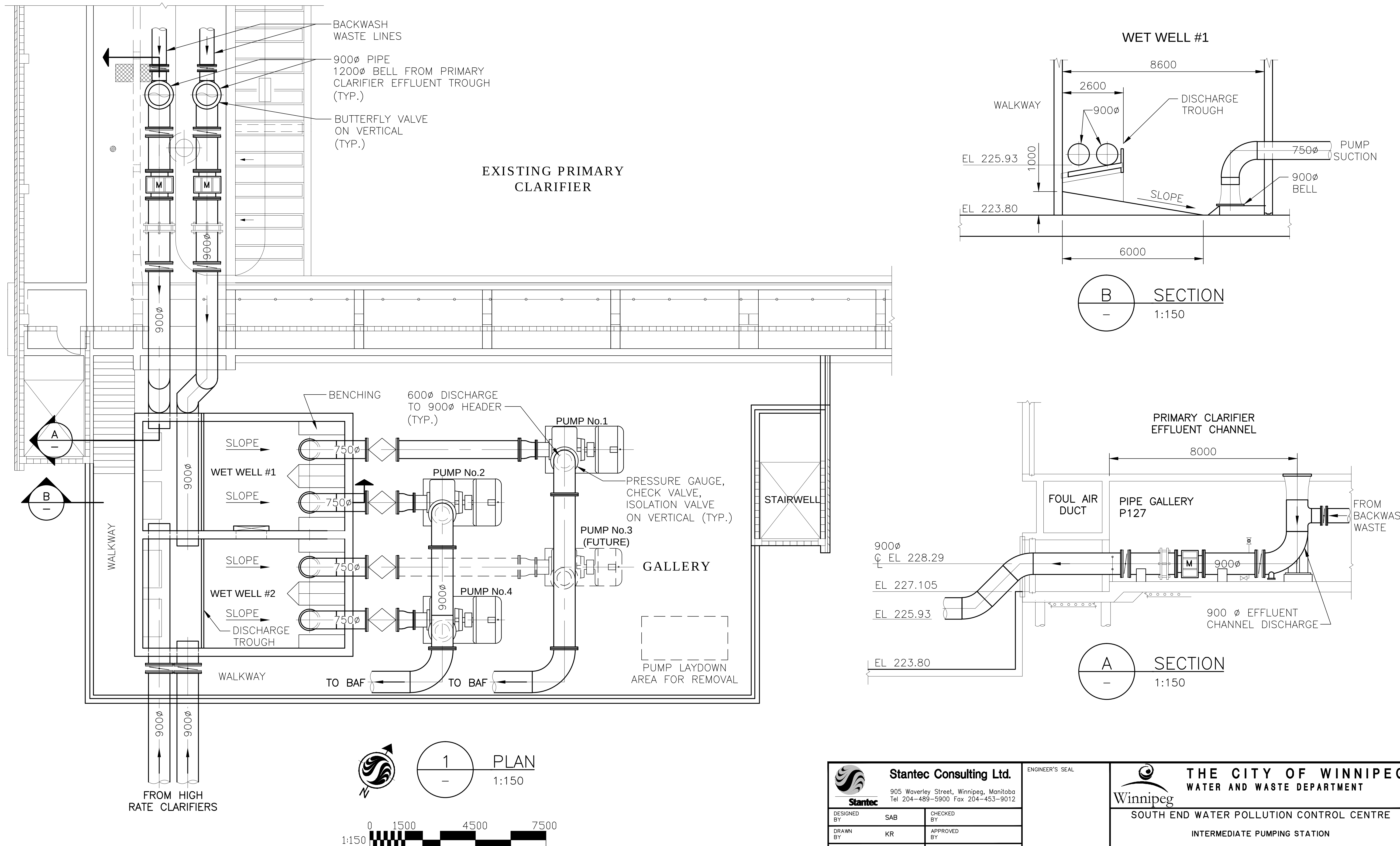

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DATE	2011.11.02	DATE	

ENGINEER'S SEAL

TENDER NO.

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

INTERMEDIATE PUMPING STATION

CITY DRAWING NUMBER FIGURE 7.7

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2012 March 20, 4:16 p.m.

Table 7. 3. These values are conservative relative to that published by Water Environment Federation (WEF). Since this assignment was to validate that the design would work, conservative values were chosen at this stage.

Table 7.3 - Estimated Removal Efficiencies for CEPT

Parameter	Proposed Minimum Removal Efficiency	Proposed Average Removal Efficiency
TSS	40%	50%
Total BOD_5	25%	35%
TP	30%	75%
TKN	10%	15%

As mentioned above, a full-scale stress test program is recommended with a single PST to establish the actual removal efficiencies under various flow conditions. Water Environment Federation (WEF) manual of Practice (MOP) No. 8 (WEF, 2010) suggests typical TSS removal in the range of 60 ~ 90%, Total BOD_5 removal of 40 ~ 70%, TP removal of 70 ~ 90%. The above range of removal efficiencies also matches our data from bench-scale jar testing completed in 2006 (refer to SEWPCC Upgrading / Expansion Preliminary Design Report (Stantec, 2008). This information is appended to this report in Appendix D. In addition, the stress testing will also allow the City to determine the full capacity of the existing primary clarifiers when operating without waste activated sludge co-thickening mode.

7.3.4 Chemical Feed System

The choice of chemical and chemical dose will impact the performance of the proposed CEPT. Either Alum or Ferric Chloride as a coagulant will be injected into the PST influent channel immediately downstream of the existing grit tanks. A flash mixer will be provided immediately following the coagulant injection point to mix the chemical into the sewage. Alternate locations for coagulant addition will be further reviewed in the Preliminary Design. In addition, polymer will be injected immediately ahead of the respective primary influent channel to PST-1, PST-2 and PST-3. Submerged flocculators in the channels will provide slow mixing of the polymer and the sewage/coagulant mixture to promote floc formation.

Chemical dosage based on published data (WEF MOP 8, 2010) indicates average metal salt (ferric chloride or alum) addition of 30 ~ 40 mg/L along with a maximum polymer dose of 1 mg/L. In the 2006 bench scale testing for CEPT, 40~60 mg/L of alum combined with 1 mg/L of polymer provided the best performance for TSS, BOD_5 and TP removal. A minimum flocculation time of 20 to 30 minutes (30 minutes used for minimum retention time) are necessary for floc formation and improve TSS, BOD_5 and TP removal.

Provision will be made for the addition of an alkalinity supplement (e.g., Soda ash or sodium hydroxide) should there be an impact on nitrification by the downstream BAF process. Also, too much phosphorus removal by the CEPT process can lead to nutrient deficiencies in the downstream BAF process and impact the nitrogen removal process.

Common chemical preparation and feed system will be utilized to supply both the CEPT and HRC.

7.3.5 Primary Sludge Removal System

Currently, primary sludge for PST-1 & PST-2 is scraped to the sludge collection trough located at the influent end of the PSTs by a traveling bridge. The sludge troughs are each equipped with an auger cross collector to convey the sludge to a common collection point. PST-1 & PST-2 are equipped with two variable speed pumps rated at 7 – 22 L/s that act in a duty/standby fashion. Primary sludge for PST-3 is scraped to four (4) sludge hoppers located at the influent end of the PSTs by a traveling bridge. A common sludge suction line interconnects each sludge hopper to two variable speed pumps rated at 7 – 22 L/s that act in a duty/standby fashion. The sludge pumps operate on an operator selectable time basis to maintain a primary sludge total solids concentration of 2.5 to 3%. A nuclear density meter is presently used to determine the total solids concentration. Based on feedback received from the City's operation staff, the nuclear density meter will be decommissioned. A sludge blanket meter in each of the existing PSTs was proposed in the Conceptual Design Report (Stantec, 2009) for optimal operation and in efforts to maintain the desired sludge blanket. Based on discussions with the City’s operation staff, it was also determined in 2009 that the direct current (DC) drives for the existing primary sludge pumps will require upgrading.

Due to the potential for a higher volume and density of primary sludge being produced in the CEPT as well as the quality of the CEPT sludge, a further review of the sludge pumping and conveyance system will be necessary during the subsequent design phases.

7.4 INTERMEDIATE PUMPING STATION

The current concept for the SEWPCC upgrade includes a BAF treatment system. Hydraulically, the BAF treatment system has significant headloss, ranging between 3 and 6 meters. This exceeds the head loss of the present treatment facility processes and cannot be accommodated within the existing hydraulic grade line. Additionally, the BAF treatment units are typically constructed at a high elevation relative to the ground in order to save on excavation, shoring and dewatering costs during construction.

These conditions require that an intermediate pump station is included in the design to provide adequate lift for the new hydraulic profile to successfully discharge using the existing outfall line.

In order to provide the future flexibility to implement phosphorus removal by biological methods rather than by chemical means, alternative intermediate pumps station concepts, location and piping systems were developed. Refer to section 7.9.14 for details.

7.4.1 Design Flows

YEAR 2031

The projected 2031 design flows are as follows:

- From CEPT: 150 ML/d max
- High Rate Clarifier Process: 50 ML/d max
- Backwash Clarifiers: 18 ML/d

Total projected 2031 Design Flow = 218 ML/d

Refer to Figure 6.7 for a site plan indicating the 2031 flows in and out of the intermediate pump station. The flows identified here are calculated/discussed in Section 7.4. Refer to Figure 6.2 or the flow schematic. Refer to Section 6.0 – Operational Philosophy and Section 7.6 – High rate Clarification.

YEAR 2061

For the year 2061, it was assumed that the HRC and BAF capacity would only be expanded to accommodate the increased flow. The unrestricted projected 2061 maximum week (Summer) flow was estimated to be 315 ML/d (Refer to Table 4.18). The increase in max 7-day rolling flows was accordingly estimated at 95 ML/d (i.e., 315-220), and round to 100 ML/d. On this basis, the 2061 BAF capacity was increased from 200 to 300 ML/d.. Hence, the estimated flow to the HRC under these conditions was calculated at 300 ML/d (BAF capacity) - 150 ML/d (CEPT capacity) = 150 ML/d.

The backwash waste flow for 2061 was estimated based on a proportionate increase in flow from 18 ML/d calculated for 2031.

In summary, the projected 2061 design flows are as follows:

- From CEPT: 150 ML/d max
- High Rate Clarifier Process: 150 ML/d max
- Backwash Clarifiers: 27 ML/d

Total projected 2061 Design Flow = 327 ML/d

7.4.2 Influent Flow Lines

7.4.2.1 Existing Primary Clarifier Flow

The existing primary clarifiers discharge through an effluent trough that contains a weir with a top elevation of 233.915 m. Flow from the primary clarifiers pass over the weir and into an effluent channel where it is currently drained into the High Pressure Oxygen (HPO) reactors.

The Intermediate Pump Station is proposed to be located adjacent to the primary clarifier effluent channel. Refer to Figure 7.6 - Plan for the location. The current drain lines to the HPO reactors would be dismantled and plugged. New drain lines would be installed in the bottom of the effluent channel connecting the effluent channel to the pump station wet well. These drain lines would discharge below the normal submerged level of the wet well at an invert elevation of 225.93 m. Two drain lines are proposed, each leading to a separate wet well chamber, as the wet well plan is proposed to be two chambers to allow for cleaning.

Each drain line would contain a magnetic flow meter with butterfly valves upstream and downstream. The butterfly valves would be used for both maintenance isolation and to equalize flow to each wet well chamber. Butterfly valves are traditionally not used in wastewater treatment facilities; however they are in this case proposed as they are used on the current primary clarifier effluent lines and are proven to be reliable generally. The practicality of using this style of valve should be discussed. Refer to Figure 7.7 – Section B-B for the section identifying the drain line from the primary clarifier effluent trough to the intermediate pump station wet well.

7.4.2.2 New High Rate Primary Clarifier

The new HRC effluent piping layout is less developed than that proposed for the primary clarifier effluent. Since the new HRC and existing primary clarifiers are on a similar hydraulic grade line it is possible that the HRC effluent lines could constrain the pump station hydraulics. This will have to be confirmed during subsequent design stages. However, from the calculations completed to date, the backwash clarifiers are the most hydraulically constraining in terms of pump station operating levels.

The high rate clarifier flow will need to be split. Flow from the HRC will be conveyed through two underground drain lines to the proposed intermediate pump station and will discharge below the normal submerged level of the wet well at an invert elevation of 225.93 m. Similar to the effluent pipe configuration described for the primary clarifiers, two parallel effluent lines are proposed to run from the HRC discharge to the wet well, with each line discharging to a different wet well chamber. This will permit operation even when the wet well is partially out of service for maintenance. Refer to Figure 7.7 – Plan for the schematic piping representation.

The HRC flows would be equally split in the pipe gallery. Each of the two lines in the pipe gallery would contain a magnetic flow meter with butterfly valves upstream and downstream. The butterfly valves would be used for both maintenance isolation and to equalize flow to each wet well chamber.

7.4.2.3 Backwash Clarifiers

It is proposed to convert the existing secondary clarifiers into backwash waste clarifier tanks. This is described in Section 7.5.4 of this document. The clarified effluent from these tanks would be piped back to the intermediate pump station. With some modifications to the clarifier launders, piping can be run in the existing pipe gallery to the intermediate pump station. Refer to Figure 7.6 – Plan for the schematic piping representation. The piping layout is less developed for the clarified effluent and no drawings are developed showing detailed pipe runs.

Flow from the backwash clarifier tanks would split and be run in a single 600 mm dia. line in the existing Pipe Gallery No. 3 back to existing Pipe Gallery No. 5, downstream of the primary clarifiers. It is proposed that this line would be connected into both new drain lines running from the primary clarifier effluent channel to the intermediate pump station.

These connecting lines would contain butterfly valves for both maintenance isolation and to equalize flow to each wet well chamber. It is not known at this time if these lines would require magnetic flow meters for precise flow control. It is likely that backflow prevention in the form of a check valve would be required on each line draining from the backwash effluent to the intermediate pump station wet well.

However, some assumptions were made regarding flows and pipeline sizing to determine if there was an effect on the pump station hydraulics. It has been determined that the backwash clarifiers will constrain the pump station operating levels. This was determined by reviewing the clarifier launder invert (Elevation 231.432) and the head losses associated with transferring a 18 MLD flow through a 600 mm diameter pipeline to the intermediate pump station.

See Figure 7.7 – Plan for the preliminary connection details to transfer backwash effluent to the primary clarifier effluent pipes and then to the pump station.

7.4.3 Influent Pump Station Design Concept

A single option has been developed for the intermediate pump station. This option proposes a wet pit / dry pit scenario using horizontally mounted wastewater pumps in the dry pit. Refer to Figure 7.7 – Plan for the proposed layout. Other options are also available and include:

- Submersible pumps in a wet well style
- Submersible pumps in a dry pit style
- Horizontally mounted wastewater pumps in the dry pit

The City has previously identified a preference for centrifugal wastewater pumps in a dry pit. A general dislike for submersible pumps was previously conveyed. The current SEWPCC pump configurations commonly reflect the third option and that is what is presented herein.

Consideration was given to how the design should accommodate the 2031 and 2061 flows. The proposed design concept is based on sizing influent and effluent lines for the 2061 flows as well as the wet well. Pumps and associated pump mechanical and electrical components have only been sized for the 2031 flows. However, pipelines have been sized for the 2061 flows and space has been allowed for these components. Consideration was also given for the need to shut down the effluent channel, wet well and other systems for future upgrading. It was determined that this type of major process interruption would be difficult to undertake and upgrades and increasing mechanical component sizes would be difficult, if not impossible, in future. Thus the only upgrades required to go from the 2031 flow to the 2061 flow would be the addition of another pump (including mechanical and electrical components) and reconfiguration of process control set points.

The layout of the facility will be a two-chamber wet well side and a dry pit side in the form of a pump / pipe gallery. The wet well should be designed and configured based on Hydraulic Institute guidelines to provide proper hydraulics to the pump suctions. The wet wells shall be separate chambers with an interconnecting slide gate. Isolation valves shall permit each wet well to be totally isolated for maintenance. Pump operation should be level controlled and thus level monitoring needs to be specified. A pressure sensor in a stilling well is recommended for level monitoring. The dry side of the pump station will include horizontally mounted centrifugal wastewater pumps and most of the process mechanical valves. Electrical components such as panels, the variable frequency pump drives and the PLC control will be installed in existing electrical and control rooms within the facility. Magnetic flow meters might be required on the pump discharges depending on the level of flow monitoring of incoming flows and the need to have accurate flow measurement at this point in the process. Magnetic flow meter requirements will be determined during subsequent design phases. Two forcemains are proposed to exit the intermediate pump station and transfer flow to the BAF treatment process.

Accommodation must be made for future maintenance and replacement of equipment. A crane will be required for lifting equipment (including pumps and valves) to a location where they can be lifted to ground level and loaded onto a truck. Access to the pump / pipe gallery will be by stairway and access from the primary clarifier pipe gallery.

The bottom of the wet well is proposed to be at an elevation of 223.80 m. This is required to prevent hydraulic backup of the backwash clarifiers and it will also provide adequate pump suction submergence to avoid vortexing and suction of air. The wet well size combined with the pump operating levels provides adequate operating volume for the pumps.

7.4.4 Solids Handling

With the proposed facility upgrades, very little solid material is expected at the intermediate pump station. Accordingly, no special provision has been made for large solids handling capabilities by these pumps. It is proposed to use solids handling pumps with the capability of handling up to a 175 mm solid.

7.4.5 Influent Pump Station Sizing and Pumping Details

The structural, room layout and pipe sizes for the pump station are to be designed for a year 2061 anticipated flow of 338 ML/d. This flow will ultimately be delivered through a 4 pump system whereby 3 pumps will act as duty and the forth will be a standby unit. For the 2031 flow of 218 ML/d, the flow will be delivered through a three-pump system whereby two pumps will act as duty unit and the third will be available as a standby. Space has been provided for the fourth pump and the associated mechanical and electrical components.

For the 2031 and 2061 flow scenarios, each pump is proposed to provide an equal flow and head. This allows all pumps to be the same/interchangeable thus reducing spare part requirements in order to facilitate operation and maintenance procedures. For the purpose of pump chamber sizing, pumps capable of producing a flow of 113 ML/d at a total dynamic head of 14 meters were used. These pumps are also capable of a turndown that will result in a pumping rate of 20 ML/d for overnight winter low flows. As the hydraulic profile is finalized and design progresses, new head loss calculations will be required and alternative pumps will be sourced as required.

All pumps will be controlled with Variable Frequency Drives (VFD). The operational intent is for a single pump to always be running. The VFD turndown will allow this. Other pumps will turn on, speed up or slow down, as required.

The overall volume of the wet well has been determined based on the approach of two pumps operating at full speed for 10 start/stop cycles per hour for the 2031 flow. This does not accurately represent how the pumps will operate, but is a conservative approach to wet well design that would allow manual pump operation at full speed without harming the pump components. This requires an active storage volume of 234 m^3. When the third duty pump is added to accommodate 2061 flows, the operating levels will require adjustment and the wet well will no longer provide adequate volume for 10 start/stop cycles per hour at full speed for all three pumps. The wet well volume would only be adequate for 15 start/stops per hour, which is inadequate. This is manageable due to the use of VFD's for the pumps. For comparison, standard pump stations with constant speed pumps are designed for 6 to 14 start / stops per hour, with most designers using 6 start / stops per hours. This is important as it relates to the heat generated in a pump motor when it starts. If a pump starts more than 14 times per hour, damage may occur to the motor.

The well will be split into two chambers with two pumps connected to each chamber. The pumps will be directly connected to the wet well. Thus for the 2031 flow scenario, with one wet well chamber out of service, the maximum pumping capacity would be 113 ML/d. The wet well chambers will be hydraulically connected through a normally open slide gate. The slide gate will help to equalize the level in each chamber during normal operation. Flow entering each wet well will discharge onto a trough that will control the flow, reduce turbulence, and improve the inlet hydraulics to the pumps.

7.4.6 Lift Station Pump Control Strategy

A proposed control strategy for the pumps has been determined and is dependent on the level in the wet well as it pertains to the active volume (minimum volume of liquid required to minimize pump start/stops to 10 cycles per hour). Each pump chamber has an active volume in excess of 117 m^3, which equates to a wet well height of 3100 mm. Additionally the pumps require a minimum submergence of 2350 mm, meaning the entire height of the wet well exceeds 5350 mm. The NPSH for the preliminary pump selection is 1.2 m. The present pump and pipe configuration exceeds this requirement.

2031 flow of 218 ML/d

Each pump will be controlled by a variable frequency drive. Pump 1 will start and operate at minimum speed when the level in the wet well is 450 mm into the active volume. As the level rises to 2000 mm into the active volume, the pump speed will ramp up to full speed. At 4650 mm, pump 1 will be at full speed and pump 2 will start at low speed. As the level rises to 6200 mm, the pump 2 speed will ramp up to full speed. Above this level, the two pumps will operate at full speed and a high level alarm will trigger at 6600 mm.

As the wet well level decreases, each pump slows and eventually stops. Pump 2 would be at full speed at 6200 mm but ramp down to minimum speed at 4650 mm and shut off at 3650 mm. Pump 1 would be at full speed at 4350 mm but ramp down to minimum speed at 2800 mm. Pump 1 should not shut off but would if the incoming flow was less than 20 ML/d. Refer to Table 7.4 for the proposed control strategy:

Table 7.4 – Proposed Pump Speed and Level Control Strategy for Design Year 2031

Component	Pump Start (low speed)	Pump at Full Speed	Pump Stop/ General Level
Wet Well Invert (Elv.= 223.80)			0 mm
Low Level Alarm			2350 mm
Pump No. 1	2800 mm	4350 mm	2500 mm
Pump No. 2	4650 mm	6200 mm	3650 mm
Pump No. 3 (Standby Pump)	Pump will not operate		
High Level Alarm (Elv.= 230.40)			6600 mm

2061 flow of 338 ML/d

Each pump will be controlled by a variable frequency drive. Pump 1 will start and operate at minimum speed when the level in the wet well is 800 mm into the active volume. As the level rises to 2350 mm within the active volume, the pump speed will ramp up to full speed. At 2400 mm, pump 1 will be at full speed and pump 2 will start at low speed. As the level rises to 3150 mm, the pump 2 speed will ramp up to full speed. At 3200 mm, pump 1 and 2 will be at full speed and pump 3 will start at low speed. At a wet well level of 3950 mm or more, all 3 pumps will operate at full speed.

As the wet well level decreases, each pump slows and eventually stops. Pump 3 would be at full speed at 3950 mm but ramp down to minimum speed at 3200 mm and shut off at 2400 mm. Pump 2 would be at full speed at 3150 mm but ramp down to minimum speed at 2400 mm and shut off at 1600 mm. Pump 1 would be at full speed at 2350 mm but ramp down to minimum speed at 800 mm. Pump 1 should not shut off but would if the incoming flow was less than 20 ML/d. Refer to Table 7.5 for the proposed control strategy.

Table 7.5 – Proposed Pump Speed and Level Control Strategy for Design Year 2061

Component	Pump Start Low Speed	Pump at Full Speed	Pump Stop/ General Level
Wet Well Invert (Elv.= 223.80)			0 mm
Low Level Alarm			2350 mm
Pump No. 1	3300 mm	4850 mm	2500 mm
Pump No. 2	4900 mm	5650 mm	4100 mm
Pump No. 3	5700 mm	6450 mm	4900 mm
Pump No. 4 (Standby Pump)	Pump will not operate		
High Level Alarm (Elev.= 230.40)			6600 mm

Full speed corresponds to a pump output of 113 ML/d. The low speed is dictated by the electrical capabilities of the motor and the hydraulic capabilities of the pumps. It can be estimated that low speed will correspond to a flow of 20 ML/d.

7.4.7 Lift Station Pumps

The preliminary pump selection for lift stations is as follows:

- Flowserve Model 24MN33B
- 113 ML/d at 14 meters of head
- Cast Iron Construction
- 150mm solids handling capability
- 600mm inlet and discharge connections
- 300 HP, 590 rpm horizontal WP1 motor

7.5 BIOLOGICALLY ACTIVE FILTRATION (BAF)

The upgrades proposed for a new secondary treatment process at the SEWPCC are based on BAF technology to accommodate the projected 2031 flows and loads to the plant.

7.5.1 BAF Process

WEF currently defines the acronym BAF as “Biologically Active Filtration” rather than “Biological Aerated Filters”, a term that has been historically used for this secondary treatment process. This change was undertaken to encompass other type of filters used in the industry, commonly referred as “denitrification filters.”

As a part of the project scope definition, two vendors were consulted. This includes John Meunier Inc. for the Biostyr® process and Degremont Technologies for the Biofor® process. Currently these two vendors have the major share of the North American market and also account for almost all BAF installations in Canada. The key features for the two systems are summarized below:

Biostyr® Process

- The process uses a floating media comprised of specially treated expanded polystyrene beads which allow growth of active biomass growth on the media. Ceiling plates with regularly spaced nozzles are provided to retain the filter media in the BAF cell.
- Upon entering the Biostyr® cells, the wastewater flows upwards through the filter media and collects in a common treated water effluent channel.
- During backwash, water from the common effluent channel flows down through the filter by gravity, thereby fluidizing the media. The “counter-current” backwashing sequence ensures efficient removal of accumulated solids. The process air grid located below the media is also used to supply scouring air during the backwash sequence.
- Depending on the effluent requirements, the system can be configured for carbon removal, nitrification and denitrification in one stage. The system can also operate in a simultaneous nitrification/denitrification mode.
- A separate tertiary denitrification BAF can also be installed to achieve a higher degree of total nitrogen removal with methanol as an external carbon source.
- The backwash waste storage tank receives the wastewater produced during a backwash. The process is typically designed to allow only one cell in backwash at any given time. The backwash storage tank acts as a flow and TSS load equalization tank, as a significant volume of backwash water is produced over a very short period of time.

Biofor® Process

- The process uses a submerged, fixed bed granular media that provides adequate support for biomass attachment and a mechanical filtration capability. A support floor above the basin grade, called a “false floor”, supports the filter media and is made of perforated concrete slabs equipped with air/water distribution nozzles. These nozzles ensure uniform distribution of the screened water during the filtration cycle and of the backwash water and the scour air during the backwashing cycle.
- Influent is introduced at the bottom of the filter, and flows upward through the filter bed designed for biomass attachment. The treated water leaves the filter over an outlet weir in a filtered water channel. The wastewater flows upwards through the filter media and then collects in a common treated water effluent channel.

- Periodic backwashing is necessary for this technology. Backwash frequency varies from 24 hours to 48 hours depending on the loadings applied and the treatment objectives. The filter wash is of the co-current type and the techniques used for washing are similar to those applied to sand filters for potable water using simultaneous water and air. The process air grid located below the media is also used to supply scouring air during the backwash sequence.

- Backwashing is achieved by pumping filtered water from the effluent well to the filter cells. An "energetic" automatic wash sequence must also be integrated in the filter wash programming and be initiated by push button, periodically (at least once per month). The wash system is generally designed to wash one cell at a time.

- Depending on the primary effluent characteristics, the system can be configured for carbon removal(C), nitrification (N) and denitrification (DN) in either two staged i.e., C/N followed by DN or three staged system i.e., C followed by N followed by DN staged. A pre-DN stage in front of the C or C/N stage also is available. The system is not designed to operate in a simultaneous nitrification/denitrification mode. For a higher degree of nitrate removal in the last DN stage, methanol is used as an external carbon source.

- Each filter is equipped with an outlet for dirty backwash water, which conveys it to dirty backwash water storage tank common to each filter battery. The dirty backwash water channel is automatically isolated by two valves during filtration cycles. Dirty backwash pumps transfer water to a separate dirty backwash water treatment unit for further processing.

7.5.2 Key Design Criteria

This section summarizes the key assumptions for the conceptual sizing of the BAF. The sizing is based on Summer season – Maximum Week flows and loadings. This condition was selected as it provides a sensible interpretation of effluent ammonia averaging period to adequately protect aquatic life.

<u>Design Flows and Loads (primary effluent)</u>

- Design flow = 200 ML/d (based on 150 ML/d from CEPT and 50 ML/d from HRC)

- BOD_5 loading = 150 ML/d * 78 mg/L+ 50 ML/d * 66 mg/L= 15,000 kg/d

- TSS loading = 150 ML/d * 75 mg/L+ 50 ML/d * 33 mg/L= 12,900 kg/d

- Ammonia loading (estimated) = 0.62 * Raw Influent TKN = 0.62 * 22 mg/L = 13.6 mg/L* 200 ML/d = 2,730 kg/d (based on no ammonia removal through either CEPT or HRC)

  - Based on a review of the historical data, it was found that the average annual fraction of ammonia relative to TKN was about 62%.

- Based on a review of the historical data, it was found that the maximum week summer TKN was about 22 mg/L-N.
- For the purposes of this analysis it was assumed that the influent did not contain any appreciable amount of nitrates. As such influent TKN = TN for all intents and purposes.

- TP loading = 150 ML/d * 0.85 mg/L+ 50 ML/d * 0.68 mg/L= 157 kg/d
- Minimum summer temperature = 12.9 ^{o}C

Based on a capacity of 200 ML/d, the proposed effluent quality requirements from the BAF are as follows (to meet the August ammonia loading):

- $cBOD_5 \leq 15$ mg/L
- TSS ≤ 12 mg/L
- Ammonia-N ≤ 1.5 mg/L
- Total Nitrogen ≤ 12 mg/L (operation objective on a 30-day rolling average)
- TP ≤ 0.8 mg/L (operation objective on a 30-day rolling average)

For maximum week summer conditions and based on the associated mass balance of flows and ammonia concentrations (refer to Table 4.18), the predicted blended effluent ammonia loading is 580 kg/d based on 14 mg/L at 20 ML/d through HRC and 1.5 mg/L at 200 ML/d in BAF effluent.

7.5.3 Preliminary Process Sizing

Both Biostyr® process and the Biofor® process take a different approach in sizing the BAF cells. The Biostyr® approach assumes a first stage simultaneous C/N/DN followed by a second stage DN only design. During the months of August and September, the first stage is operated primarily as a C/N stage to meet the stringent effluent ammonia loads of 607 kg/d and 703 kg/d respectively. Hence, a seasonal switchover mode will be required if the Biostyr® process is selected. In discussions with the vendor, it was confirmed that this switch over from a C/N/DN mode to C/N mode could be completed in a matter of hours. There is currently no operating Biostyr® installation in North America that operates on a simultaneous C/N/DN mode.

For the Biofor® process, a more traditional approach was undertaken by the vendor. This approach relies on a three-stage design consisting of first stage C followed by a second stage N and ultimately a third stage DN to meet the proposed effluent criteria. The vendor indicated that while this is a conservative approach, it allows better process optimization of individual stages and does not require any switch-over mode for meeting the seasonal ammonia limits. The project definition document is based on a three-stage Biofor® process (i.e., followed by N followed by DN stages). A summary of the concept is provided as follows:

- Total number of cells required: 24
  - C stage: 8
  - N stage: 8
  - DN stage: 8
- Total Surface area of cells: 1760 m^2
  - C stage: 652 m^2
  - N stage: 861 m^2
  - DN stage: 247 m^2
- Depth of filtration media: 3.9 m
- Media support gravel: 0.3 m
- Backwash pumps (duty/standby)
- Backwash air blowers (duty/standby)
- Process air blowers (duty/standby dedicated for C and N stages)
- Media size (mm): 2.7
- Total headloss through the process (approx.): 5.7 m
- The system is designed with one (1) cell under backwash i.e., off-line

7.5.4 Backwash Waste Management

7.5.4.1 Backwash Waste Flows and Characteristics

The sludge produced by the BAF is managed through a filter backwash. The TSS concentration in the dirty backwash water typically varies from 500 mg/L to 1,500 mg/L depending on the type of treatment (i.e., C, N or DN), cycle time, and water used. Based on discussions with the vendor, an assumed a flow of 16,000 m^3/d (summer maximum week basis) for backwash waste along with a maximum TSS concentration of 1,500 mg/L has been used in the design estimates.

Each filter is equipped with an outlet for dirty backwash water, which conveys it to a dirty backwash waste holding tank common to each filter battery. Preliminary estimates indicate that this tank will require an approximate operating volume of 3,500 m^3 based on handling daily backwash flows from the largest cells i.e., C and N simultaneous. The dirty backwash water channel is automatically isolated by two valves during filtration cycles. Dirty backwash pumps will transfer the dirty backwash to the proposed clarification unit utilizing the existing circular secondary clarifiers.

7.5.4.2 Existing Secondary Clarifiers

The SEWPCC is currently equipped with three (3) center column siphon feed and peripheral overflow type secondary clarifiers. Each has a central bridge driving mechanism that supports and rotates a center cage with two sludge rake arms and two scum blades.

Currently, mixed liquor from the HPO bioreactors is conveyed to Secondary Clarifiers No. 1, 2 and 3 via influent pipes. Each influent pipe is fitted with a magnetic flow meter and butterfly control valve to regulate and record the flow of mixed liquor, as shown in the adjacent figure. Effluent overflows a V-notch weir into a circumferential launder around the perimeter of each clarifier. The effluent in the launder drains to a conduit that discharges into an effluent drop shaft. Clarifiers No. 1 and 2 share a common drop shaft while Clarifier No. 3 drains to a separate drop shaft.

The dropshafts discharge to the plant bypass channel that spans the length of the secondary clarifier area. The plant bypass channel conveys both secondary effluent and any raw or primary effluent bypass flows from the upstream processes to the ultraviolet (UV) disinfection system.

7.5.4.3 Proposed Modification of the Secondary Clarifiers for Backwash Clarification

It is proposed that for the proposed concept of chemical phosphorus removal followed by BAF, the existing secondary clarifiers will be used for backwash clarification. It should be noted that if biological phosphorus removal is implemented in the future, the existing clarifiers will function as secondary clarifiers and a new system will be developed for backwash waste treatment. The three (3) clarifiers can currently handle a maximum flow of 100 ML/d of mixed liquor suspended solids (MLSS) ranging from (2,000 ~ 3,000 mg/L) from the HPO process. Clarifiers 1 and 2 each have an area of 880 m^2 and clarifier 3 has an area of 1,640 m^2.

Three options were evaluated as follows:

- Option 1: Use Clarifiers 1, 2 and 3
- Option 2: Use Clarifiers 1 and 2
- Option 3: Use Clarifier 3 only

Based on design backwash water flow of 18,000 m^3/d or 18 ML/d, the calculated overflow rates are summarized in Table 7.6 below.

Table 7.6 – Summary of Dirty Backwash Water Clarification Design

Option	Option 1	Option 2	Option 3
Clarifier Dimensions			
Diameter (m)	33.5 (1 and 2), 45.7 (3)	33.5 each	45.7
Side Wall Depth (m)	4.6	4.6 each	4.6
Volume (m^3)	4048 (1 and 2), 7544 (3)	4048.0 each	7544.0
Surface Area (m^2)	880 (1 and 2), 1640 (3)	880.0 each	1640.0
Weir Length (m)	105 (1 and 2), 144 (3)	105 each	144
Flow Distribution (ML/d)			
At 18 ML/d	4 each (1 and 2), 8 (3)	8 each	16 (each)
Percentage of Total Flow	25% (1 and 2), 50% (3)	50% each	100% (each)
Surface Overflow Rate (SOR)			
At 18 ML/d	5.3 $m^3/m^2/d$	10.2 $m^3/m^2/d$	10.9 $m^3/m^2/d$
Weir Loading Rate			
At 18 ML/d	50.8 $m^3/m/d$	85.7 $m^3/m/d$	125 $m^3/m/d$

* Note : Values shown in brackets refer to clarifier number

Based on the above analysis, Option 2 is recommended. The SOR for Option 2 and 3 are similar; however Option 2 provides an additional benefit of 50% redundancy in operation of the facility. The Calculated SOR is well within the Manual of Practice No. 8 (WEF, 2010) for an average design SOR of 18 $m^3/m^2/d$ and a peak of 40 $m^3/m^2/d$ limit. Similarly the Weir Loading Rate is within the recommended operating value of 250 $m^3/m/d$. The solids loading rate (SLR) was not calculated as the level of solids in the backwash waste is significantly lower than the MLSS currently handled by the existing clarifiers. Also, BAF solids generally have good settling properties and with chemical enhancement provided, the proposed system has sufficient capacity to treat the backwash waste from the BAF process and provide additional phosphorus removal. Clarifier 3 can be used as a stand-by unit when either of the smaller clarifiers are taken out for maintenance or designated for future use when the BAF process is expanded.

Following clarification, the water will be directed back to the BAF via the proposed intermediate pumping station. The common drop shaft associated with Clarifier 1 and 2 will be blocked and a new piping connection will be made with the common effluent channel from the clarifiers allowing the clarified effluent to be diverted to the wet well of the proposed pump station by gravity. Further details will be developed and additional options will be evaluated in the preliminary design phase of the project.

7.6 HIGH RATE CLARIFICATION (HRC)

The proposed design of the high rate clarification (HRC) process required to provide primary treatment during wet weather events is as follows:

7.6.1 HRC Process

High rate clarification is defined as a physical-chemical system that achieves enhanced particle settlement through the use of chemicals, a ballasting agent for flocculation (e.g., microsand or chemically enhanced sludge) followed by lamella clarification. The process is well suited to provide additional primary clarifier treatment during wet weather events due to the high level of suspended solids removal, robustness and fast startup. HRC is reported to reduce TSS from 70 to 95 percent and BOD_5 from 60 to 65 percent (A. Szekeress •Treating Wet Weather Flows with Biologically and Chemically Enhanced High Rate Settling, September 21, 2010). In addition, the proposed system will also remove total phosphorus and particulate nitrogen. The percentage removal of various constituents are shown in Table 7.8.

For the SEWPCC upgrades, plant flows from 150 ML/d to 325 ML/d will be directed to the HRC. HRC effluent up to 50 ML/d will be diverted to the BAF via the intermediate pump station. HRC effluent from 50 ML/d to 175 ML/d (total 125 ML/d) will be directed to chlorination/dechlorination.

As a part of the project scope definition, two vendors were consulted. The vendors and respective processes are Veolia Water for the Actiflo® process and Degremont Technologies for the Densadeg® process. Currently these two vendors have a major share of the market and also account for almost all HRC installations in North America. The key features for the two systems are summarized below:

<u>Actiflo® Process</u>

- Process relies on the use of microsand to enhance flocculation and settling and includes four (4) distinct cells (Coagulation Tanks, Injection Tank, Maturation Tank and Settling Tank).

- Coagulant Tank: Micro-flocculation occurs in this cell. A coagulant is added to the influent prior to reaching cell.

- Injection Tank: Microsand and polymer are injected into this mixed cell.

- Maturation Tank: High efficiency mixing created by the use of a draft tube allows the microsand/sludge floc to agglomerate and grow into high density floc.

- Settling Tank: Dense microsand/sludge floc settles rapidly to the bottom of the tank. Settling is further enhanced by the use of lamella tub settlers. Microsand/sludge settled at the bottom of the settling tank is continuously recycled back to a hydrocyclone where microsand is separated from the sludge. Sludge is wasted and the recovered microsand is returned to the Injection Tank.

- In comparison to the Densadeg® process, the Actiflo® process produces a higher volume, lower concentration sludge. Additional thickening is necessary.

- Actiflo® startup is quicker than the Densadeg®.

Densadeg® Process

- Process relies on the use of recycled sludge to enhance flocculation and settling and includes three (3) distinct zones (Rapid Mix Zone, Reactor Zone and Settling Zone).

- Rapid Mix Zone: Coagulation occurs in this zone when a coagulant is rapidly mixed with the influent to destabilize particles so that they will more readily agglomerate and as a result form larger floc.

- Reactor Zone: Polymer and recycled sludge injected in this zone is mixed with the influent in an axial flow turbine to promote solids contact and dense floc formation.

- Settling Zone: The dense floc rapidly settles to the bottom of the settling zone while tube settlers are used as a polishing step to ensure lighter, low density floc also settles to the bottom. Settled sludge is thickened using a rotating scraper and a portion is recycled back to the Reactor Zone. The remainder of the sludge is wasted via an automated sludge blowdown valve.

- In comparison to the Actiflo® process, the Densadeg® process produces a lower volume, higher concentration sludge.

- Densadeg® startup is slower than the Actiflo®.

While each system is unique in terms of the components internal to the tanks, overall tank dimensions and sludge concentrations, they are similar in terms of their configuration and layout on the site. In an effort to be conservative, the Densadeg® process has been used for purpose of site layout, as the proposed footprint is approximately two times that proposed for the Actiflo® process. The Actiflo® process has been used in determining the side stream hydraulic profile, as it has a higher headloss. The Actiflo® process produces a less concentrated sludge at a higher rate than the Desadeg® process and therefore the biosolids handling system will be based on the Actiflo® process.

7.6.2 Proposed High Rate Clarification System

The following design criteria have been assumed for preliminary sizing of the high rate clarification process. The flows and loads shown in Table 7.7 are the portion going to the HRC after 150 ML/d has been directed to the CEPT. For the purpose of this study the criteria for HRC effluent quality was developed based on published data and discussions with vendors.

Table 7.7 – HRC Influent Design Criteria

Season	Design Period	Flow (ML/d)	TSS Loading (kg/d)	BOD_5 Loading (kg/d)	TKN Loading (kg/d)	TP Loading (kg/d)
Spring	Max Week	51.0	8,272	8,227	1,356	240
	Max Day	150.0	53,635	24,788	3,633	614
Summer	Max Week	70.0	11,585	9,249	1,535	237
	Max Day	175.0	37,036	26,043	3,976	811
Fall	Max Week	4.0	504	558	125	17
	Max Day	122.1	17,799	14,294	2,606	462

Table 7.8 – HRC Removal Efficiencies

Parameter	Minimum Percent Removal	Average Percent Removal
TSS	70%	85%
BOD_5	50%	60%
Particulate TKN	35%	40%
TP	65%	80%

Both vendors propose two 50% units to accommodate the peak design flows. The units will be constructed in parallel and will be separated by a common pumping and piping gallery along the long axis of each tank. A splitter box will be used to evenly split flow between each tank when flows are greater than 50% of the peak flow. When flows are less than 50% of the peak flow only one tank will be used. Automatic weir gates installed in the splitter box will be used to control single tank versus dual tank operation.

The tanks will be covered by a superstructure to mitigate against freezing during the spring and fall periods when liquid is stagnant in the tanks. An electrical room, mechanical room and office are proposed for construction adjacent to the HRC tanks.

The chemical room will include a coagulant dosing system and a polymer dosing system. The coagulant (typically either ferric chloride or alum) will be determined by the selected vendor through pilot or bench-scale testing. The coagulant system will be comprised of double walled storage tanks sufficient to accommodate 30 days storage at the projected average day flow. The coagulant dosing system will be located in a chemical room adjacent the existing primary clarifiers. This chemical room will be common to the CEPT process.

Dry polymer is preferred over liquid polymer as it is significantly less expensive. Dry polymer also requires time to mature (typically 60 minutes) and the HRC system is required to start-up quickly in reaction to increased wet weather flow. As such, it is proposed to rely on the CEPT polymer make-up system to provide polymer to HRC polymer dosing system, as the CEPT process requires polymer at all times. The polymer feed system will be located in a chemical room adjacent to the existing primary clarifiers. This chemical room will be common to the CEPT process.

7.7 EFFLUENT DISINFECTION

The Licence for the SEWPCC stipulates that the fecal coliform and Escherichia coli (E. coli) content in the effluent shall be less than or equal to 200 MPN (most probable number) / 100 mL, as determined by the monthly geometric mean (GM) of 1 grab sample collected at equal time intervals on each of a minimum of 3 consecutive days.

All flows up to 325 ML/d (peak day) will be disinfected. The proposed design splits flows between the main stream primary clarification followed by BAF and the side stream HRC. The BAF is rated up to 200 ML/d and the BAF effluent will be disinfected using UV light. The flows greater than 200 ML/d, but less than 325 ML/d, are treated by the HRC process and will be disinfected using chlorination/dechlorination. The portion of flows in excess of 325 ML/d will not receive disinfection.

7.7.1 Impact of Bypassing Disinfection at Peak Flows

An analysis was performed to estimate the monthly GM of E. coli concentrations in the final effluent based on projected wet weather flows for the design horizon of the year 2031 and the level of disinfection required to comply with microbial limits established by the SEWPCC Licence. It has been assumed that both the disinfection systems noted earlier will be designed to achieve E. coli concentrations below 200 MPN/100 mL, to allow for infrequent wet weather E. coli excursions in the final effluent in response to the blending of untreated flows in excess of 325 ML/d to be in compliance with the microbial effluent limit.

The stress pattern that has been developed as part of Section 6 has been used to estimate the number of hourly wet weather events that will exceed the maximum day flow of 325 ML/d in order to approximate the blended effluent E. coli concentrations. The stress pattern was developed using six years of flow data from the SEWPCC. It has been assumed that the concentration of E. coli in the raw wastewater will be in the order of 10^6 to 10^7 MPN/100mL (MacLaren Engineers Inc., 1986). Further, it has been assumed that after dilution with wet weather flows and the reduction associated with screening and grit removal, the concentration of E. coli in the non-disinfected effluent that is to be blended with the disinfected effluent will be reduced by 75 percent (Metcalf and Eddie, 2003). This information has been used to assess the number of events per month that would exceed 200 MPN/100mL and average dry weather E. coli levels required on a daily basis to achieve a monthly GM mean of 200 MPN/100mL, or less. The current Environment Act Licence requires daily sampling when the flow exceeds the 75th percentile of the thirty-day average maximum daily flow, and samples are to be analyzed for E. coli content. In addition, the Licence requires the GM is to be based on a minimum of 3 grab samples at equal time intervals per week. In this analysis, the City's current practice of sampling 7 days per week has been assumed to determine compliance with E. coli limits. If the disinfection systems were both operated to achieve exactly the 200 MPN/100mL limit at all times for flows up to 325 ML/d, the assessment predicts that there will be a maximum of 16 days over a 6 years that will exceed a 200 MPN/100mL limit. This results in 13 months that would not meet a monthly E. coli GM of 200 ML/d over a six-year period as depicted in Figure 7.8. An analysis was done to estimate the target effluent concentrations for E. coli from both disinfection systems so that compliance could be achieved for a monthly GM limit of 200 MPN/100mL. The analysis found for a raw wastewater E. coli concentration of 10^7 MPN/100mL, the disinfections systems would need to be operated to achieve an effluent E. coli concentration at or below 150 MPN/100 mL on an average daily basis.

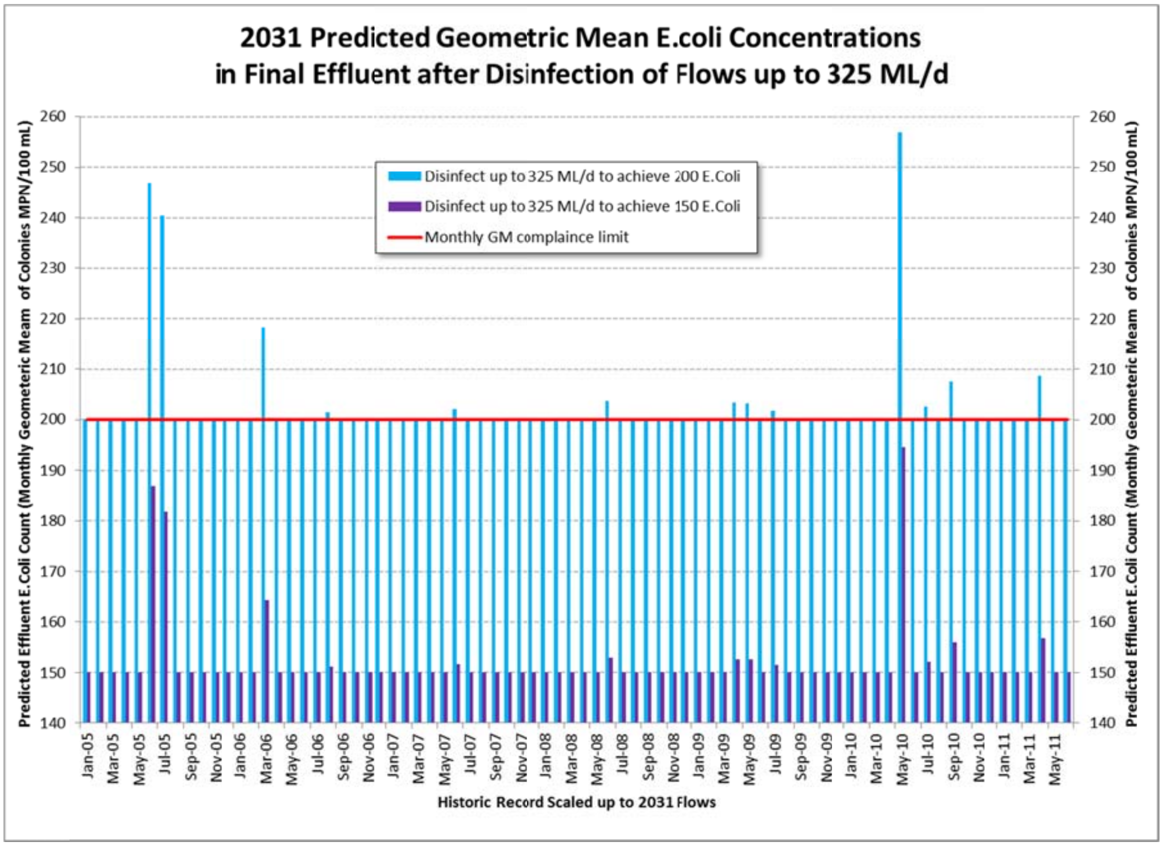


Figure 7.8: 2031 Predicted Geometric Mean E.coli Concentrations in Final Effluent after Disinfection of Flows up to 325 ML/d

7.7.2 UV Disinfection System

The existing UV disinfection system was designed to disinfect flows less than 100 ML/d and was only intended to operate when Red River levels measured at the outfall are less than 229.00. The current licence only requires UV disinfection to take place when the Red River elevation measured at the outfall is less than 229.00. The design criteria for the existing UV system are summarized in Table 7.9.

Table 7.9 – Existing UV Disinfection System Design Criteria

Description	Value
Peak Hydraulic Capacity	100 ML/d
Maximum Total Suspended Solids	10 mg/L
Minimum UV Transmittance	50%

The existing UV disinfection facility includes two (2) parallel 1.4 m wide UV channels that each include two (2) banks per channel. Each bank includes 5 modules and each module has 6 bulbs for a total of 30 bulbs per bank.

Currently the UV disinfection facility is effective when overall plant flows are less than 100 ML/d and can be non-compliant when overall plant flows exceed 100 ML/d. When plant flows exceed 100 ML/d the current hydraulic configuration allows for mixing of primary and secondary effluent before the combined effluent is directed to the UV disinfection facility. An analysis of the SEWPCC disinfection results from January 2011 to September 2011 indicates that the average E. coli concentrations are approximately 30 MPN/100mL.

The new licence conditions require the UV system to operate year round under all river levels. To determine if this is possible by expanding the existing UV facility, Stantec undertook hydraulic analysis via numerical modeling. Refer to Section 6.11 for more details regarding the hydraulic model. It was determined that with a peak flow of 200 ML/d through an expanded UV system and no flow through the side stream bypass the UV system could disinfect flows up to a river level of 229.7 m. With the peak flow of 420 ML/d (200 ML/d through the expanded UV system and 220 ML/d bypassing the UV system) the maximum river level where disinfection is still effective is 226.8 m. Refer to Table 7.10 for Red River return periods at the SEWPCC outfall location.

Table 7.10 – Red River Elevations at the SEWPCC Outfall and Return Period

Return Period	River Level at SEWPCC Outfall
5	228.1
10	228.4
20	228.4
33	228.7
50	229.8
100+	231.2

Source: Grant Mohr, City of Winnipeg, Water and Waste Department, July 17, 2006.

Based on the Red River elevation return periods, UV disinfection using the existing system configuration will not be effective under peak flow at river levels less than the five year return period. Flows to the expanded UV system will be from the BAF only and as the influent to the BAF is pumped, it is proposed to construct a new UV disinfection facility at a higher elevation. The proposed elevation was selected based on disinfection being effective at peak day flows (325 ML/d) and the UV facility will not flood under the peak instantaneous flows (420 ML/d) at a river level of 230.00, which is in excess of the 50 year return period.

Hydraulic modeling determined that the new UV disinfection facility would be constructed 1.66 meters higher than the existing UV facility to achieve the conditions listed above. With the UV facility set at this elevation disinfection would be effective for the total pumped flow of 420 ML/d at all times when the river level is less than 228.72 m.

The proposed UV facility design is based on the Trojan 4000 UV Plus technology by Trojan Technologies. The proposed design allows for only effluent from the BAF to be discharged to the UV disinfection facility. The hydraulic capacity of the BAF is 200 ML/d and therefore the UV system must be designed to accommodate a peak flow of 200 ML/d. The design criteria for the proposed UV disinfection system are summarized in Table 7.11.

Table 7.11 – Proposed UV Disinfection System Design Criteria

Description	Value
Peak Hydraulic Capacity	200 ML/d
Peak Total Suspended Solids	10 mg/L
Minimum UV Transmittance	50%
Max Average Particle Size	30 microns
Max E. coli concentration in effluent	200 MPN/100 mL
Average E. coli concentration in effluent	< 150 MPN/100 mL

Four (4) new channels will be constructed in parallel. The equipment from the existing UV system will be relocated to two of the new channels and new equipment will be installed in the other two new channels. The new channels will each be 1.4 m wide and will include two (2) banks per channel. Each bank will include 5 modules of 6 bulbs per model for a total of 30 bulbs per bank. A new superstructure will be constructed to enclose the new channels and electrical room.

7.7.3 Chlorination / Dechlorination System

The proposed design passes wet weather flows greater than 200 ML/d, but less than 325 ML/d through the side stream vortex grit removal and HRC process. The Process Selection Report (Veolia, 2011) identified that wet weather flows will be disinfected using chlorine. For the purposes of this evaluation and discussion, chlorination and dechlorination are to be considered an explicitly linked treatment system for disinfection. The proposed design assumes that the existing high purity oxygen (HPO) reactors will be retrofit to provide the required chlorine contact time for disinfection. The chlorination/dechlorination design is based on Ontario's Ministry of Environment (MOE) Design Guidelines for Sewage Works (2008) and assumes the design criteria identified in Table 7.12.

Table 7.12 – Proposed Chlorine Disinfection System Design Criteria

Description	Value
Peak Hydraulic Capacity	125 ML/d
Contact Time at Peak Flow	15 minutes
Length to Width Ratio	10:1 (min)
Height to Width Ratio	2:1 (max)
Baffling Factor	0.7
Volume Required	1,860 m^3
Max E. coli concentration in effluent	200 MPN/100 mL
Average E. coli concentration in effluent	< 150 MPN/100 mL

It is proposed to use existing HPO Reactor No. 4 to buffer smaller wet weather flow events and existing HPO Reactor No. 3 to provide for chlorine contact time for disinfection of the HRC effluent. Small wet weather events that result in less than 1,940 m^3 could be stored in Reactor No. 4 and then drained back to the intermediate pump station when the wet weather event has subsided. When the wet weather event exceeds 1,940 m^3, the HRC effluent would pass to the chlorine contact chamber. The HRC effluent would be dosed with chlorine in a completely mixed chamber before traveling though serpentine channels to achieve the appropriate contact time. Sodium bisulfate would be added to the effluent at the end of the chlorine contact chamber in a completely mixed zone for dechlorination before being discharged to the outfall via the proposed plant bypass.

It has been assumed that on-site sodium hypochlorite generation will be used for chlorine disinfection to remain consistent with the current practice at the new Water Treatment Plant. It has been assumed that the existing HPO Equipment Room can be reused to locate the on-site sodium hypochlorite generation equipment. The design will include a monitoring system that will stop chlorine addition if the dechlorination system fails to mitigate against the discharge of chlorinated effluent to the river.

There are some health, safety, and environmental risks associated with the use of a chlorination and dechlorination system for the disinfection of wet weather flows. Mitigative control measures and operational procedures will be reviewed and implemented to eliminate or substantially reduce the risks associated with this technology for intermittent wet weather disinfection.

7.8 BIOSOLIDS

The general biosolids handling approach was previously presented in Section 5 and is further defined as follows:

7.8.1 Background

The inclusion of onsite solids processing is an important consideration in the design of the liquid train treatment processes. The Program Team has not selected a Biosolids Master Implementation Plan for the City's three wastewater treatment facilities. The selected biosolids implementation strategy has the potential to impact on loads at the SEWPCC significantly. The development of an overall Master Plan would reduce the risks associated with advancing the project definition for the SEWPCC.

As part of the SEWPCC Project Definition work plan Stantec have assumed that anaerobic digestion and dewatering will be located at the SEWPCC and included in the overall plant design. For this scenario, Stantec was instructed by the Program Team that biosolids from the WEWPCC will continue to be processed at the NEWPCC.

The proposed chemical phosphorus removal treatment process at the SEWPCC will result in an increase in sludge mass and amount of inert solids in the sludge. This has the potential to negatively impact the size and performance of an anaerobic digestion process to be located at the SEWPCC.

A preliminary mass balance was performed on the plant processes to assess the impact of including solids treatment at the SEWPCC particularly on the influent loading to the liquid treatment trains. Specifically, a mass balance analysis was done based on PSR Option 4 and expanded to include anaerobic digesters and biosolids dewatering, as well as an estimation of the reject waters requiring treatment.

7.8.2 Biosolids Handling Processes

The sludge generated CEPT, HRC and backwash waste clarification will be collected in a blend tank and thickened to a target solids concentration of 5%. A 2-stage anaerobic digestion process is assumed to meet volatile solids reduction criteria for stabilized sludge (biosolids). Following the digestion process Stantec have assumed sludge holding tanks will provide a total of seven (7) days storage prior to centrifuge dewatering. Dewatered biosolids will be temporarily stored in silos (similar to the NEWPCC dewatering building), from where they will be discharged by gravity to trucks for final disposal off-site.

7.8.3 Sludge Blend Tank

A sludge blend tank is provided to accommodate sludge generated by the plant at the maximum week summer flow of 220 ML/d. Preliminary estimates indicate that a tank with 2 ML active volume will be required. As indicated above, the tank will receive sludge from CEPT, HRC and backwash waste clarifiers. Two (2) tanks, each with a capacity of 1 ML will be provided. The tanks will have submersible mixers to ensure complete mixing prior to being pumped to the sludge thickeners. The blended solids concentration is estimated at 3% total solids (dry weight basis).

7.8.4 Sludge Thickening

The thickener is needed to optimize the size of anaerobic digesters. The blended sludge at 3% total solids will be thickened to 5% total solids prior to anaerobic digestion is 5%. Two (2) duty and one (1) standby drum thickeners are assumed, each rated to handle a solids loading of 864 kg/hr.

7.8.5 Anaerobic Digestion

2-stage anaerobic digestion is assumed to meet the volatile solids reduction criteria for stabilized sludge (biosolids). The digestion process was sized based on the following criteria:

- Maximum month volatile solids of 28,846 kg/d (based on 82% of TSS)
- Specific gravity of TSS = 1.03
- Solids Retention Time = 20 days
- Volatile Solids (VS) Loading Rate = 2.2 kg VS / m^3d
- Safety Factor = 1.1 (based on grit accumulation and foaming)

Based on preliminary estimation of solids generation the following concept for anaerobic digestion has been developed:

- A total of four (4) digesters consisting of 2 primaries and 2 secondary, each having maximum operating volume of 4.9 ML.
- Two (2) primary digesters approximately 27.5 m in diameter, completely mixed with fixed cover. Maximum liquid depth is 8.2 m.
- Two (2) secondary digesters approximately 27.5 m in diameter, stratified with floating cover. Maximum depth is 8.2 m.

- For maintenance, only one digester will be taken down at any given time. The piping network will be set-up to cross connect either primary digester to either secondary digester.

7.8.6 Dewatering

Digested sludge is conveyed from the anaerobic digesters by gravity to two (2) sludge holding tanks (SHTs) each approximately 20 m in diameter. This will provide 7 days of storage prior to dewatering. For the purpose of concept development, it was assumed that sludge will be pumped from the sludge holding tanks to the dewatering process, which consists of chemical conditioning and centrifuges. Centrate removed from the sludge dewatering process is diverted to the centrate treatment. The wet cake is assumed to have a solids concentration of 25% which will be easier to haul for disposal.

It was assumed that dewatering will occur over an 8 hour day, five days a week. Two (2) duty and one (1) standby centrifuge, each rated at 42 m^3/hr, will be used to dewater the digested sludge.

7.8.7 Centrate Treatment

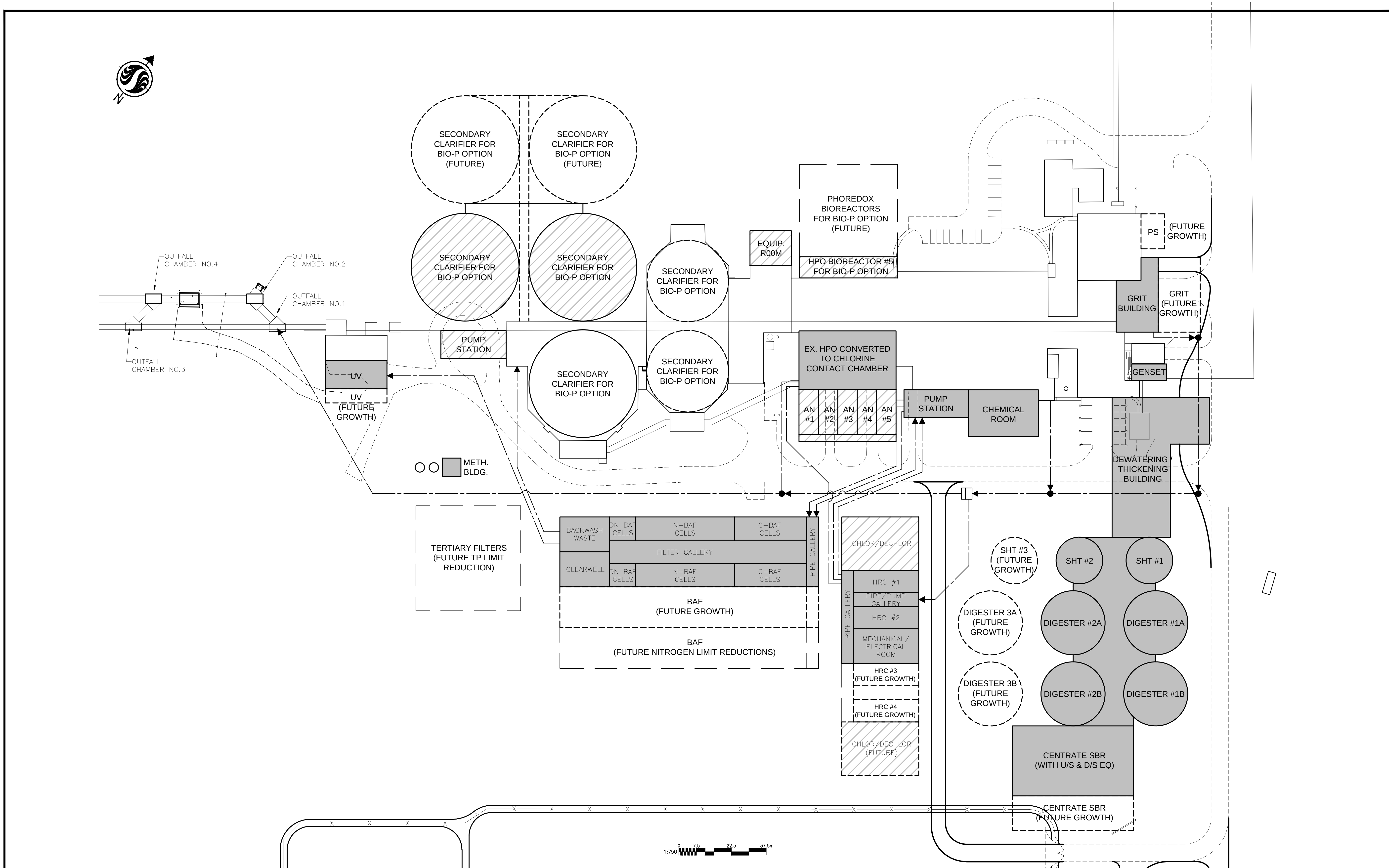
As part of the concept development, alternative technologies for centrate treatment were not evaluated. In discussion with the Program Team, Stantec assumed a facility similar to the North End Water Pollution Control Centre (NEWPCC) Centrate Treatment System. This includes a Sequencing Batch Reactor (SBR) based process train for nitrification and denitrification with methanol. The treated effluent will be returned to the head of the plant. The optimal return point for the treated centrate will need to be addressed in subsequent engineering analyses. Upstream equalization is required to allow time for mixing with thickener filtrate to cool the centrate and attenuate influent loads. For the purpose of this report an assumed temperature of 38°C for the centrate and 15°C for filtrate was used. Equalization ahead of centrate treatment is recommended to attenuate influent loads. Equalization of treated effluent and its discharge back to the headworks will be reviewed for process optimization and balancing reasons during the schematic design.

Based on mass balance presented in Section 5.0, the design criteria for the centrate SBR is as follows:

- Design flow = 1.2 (max month spring) ML/d (avg), 1.5 ML/d (max days)
- Two SBR Cells = 2,900 m^3/per cell

7.9 SITE DEVELOPMENT

This section provides the rationale for the conceptual site layout presented in Figure 7.9. The layout presented identifies between components required to accommodate the proposed 2031 design, expansion required to accommodate population growth projected up to 2061 and expansion to accommodate potential future effluent requirements. Refer to Section 3 for the project flows for 2061. Future loads for 2061 were estimated by extrapolating the loads



LEGEND

▬ (grey)	2031 (20 YR)
▨ (hatched)	NEW CONSTRUCTION FOR BIO-P OPTION
- - - -	2061 (50 YR)
—— ——	TP & TN LIMITS
SHT	SLUDGE HOLDING TANK
HRC	HIGH RATE CLARIFICATION
BAF	BIOLOGICALLY AERATED FILTERS

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO.	REVISIONS	DATE	BY

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 Fax 204-453-9012

DESIGNED BY	J.B.	CHECKED BY	J..B
DRAWN BY	K.R.	APPROVED BY	S.K.B.
HOR. SCALE:	1:750	RELEASED FOR CONSTRUCTION:	
VERTICAL:	–		
DATE	2011.11.30	DATE	

ENGINEER'S SEAL

TENDER NO.

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

SITE PLAN

CITY DRAWING NUMBER: FIGURE 7.9

SHEET OF

2012 March 20, 4:19 p.m.

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calculated for the 2031 population to the population assumed for 2061. Refer to Figure 6.2 for the flow schematic for the proposed 2031 design. The flow schematic identifies various locations where the flow is split between the main process stream and the wet weather side stream. The design criteria for each unit process have been defined previously in this section.

7.9.1 Land Availability

The SEWPCC is located on Registered Plan No. 10530 south of the Perimeter Highway 100. The City of Winnipeg owns the 638.7 acres of land that make up Registered Plan No. 10530. The City currently leases approximately 516.6 acres for cultivation and 4.9 acres to Winnipeg Radio Control for a model airplane runway. Refer to Figure 7.10 for a site plan outlining the extents of Registered Plan No. 10530 and the areas that are currently leased for other use. The Public Works Department has also constructed a snow dump facility on 24.7 acres south of the SEWPCC. This leaves an area of approximately 93.1 acres for the SEWPCC.

The proposed expansion to accommodate 2031 design flows does not encroach on any of the leased properties, although the road around the proposed solids handling facility and the future centrate facility for 2061 design flows does encroach on the snow dump berm, fence and entrance road. The snow dump berm, fence and entrance road will require modification to permit expansion for the proposed solids handling concept. Further investigation is required in subsequent design phases after the solids handling processes have been confirmed and the design has been advanced.

7.9.2 Headworks

The headworks facility requires expansion to accommodate new vortex grit removal for peak hour flows (420 ML/d). It is proposed to expand the grit removal facility east of the existing facility so that the side stream HRC processes can be located on the south side of the site along with the main stream processes. Locating the HRC process on the south side of the site allows for the side stream effluent flows to be split between the BAF via the intermediate pump station and the side stream bypass pipe.

The proposed vortex grit system will block the south overhead door entrance to the existing screen facility. The overhead door is the only access provided to remove the screenings and grit bin. As such, the screening facility will be expanded to the south to accommodate the rotated screening bin and adding a new overhead door entrance on the east side of the screening facility. Refer to Figure 7.4 for a layout of the proposed grit building expansion.

The catch basin currently used for the recreational vehicle dump will be eliminated as it is currently located in the footprint allotted for the proposed vortex grit removal system and recreational vehicles will no longer be permitted to dump at the SEWPCC.

The South End catchment interceptor sewer system and the headworks facility will require upgrades to accommodate the 2061 peak hour flow of 680 ML/d. When the interceptor capacity is increased, it is proposed to construct a new wet well at a higher elevation than the existing wet well. The new wet well would only handle flows that result from surcharging of the existing wet well. Flows from the new wet well would be conveyed to the existing channel upstream of the screens.

Additional 6 mm perforated plate screens will be required to accommodate the 2061 peak hour flow. The number of screens required will be investigated further in subsequent design stages so that enough space is allotted within the headworks expansion to accommodate new screen and bypass channels.

The grit facility will require a second expansion to provide grit removal for flows up to 680 ML/d. It is proposed to mirror the proposed grit facility to the east by adding two additional vortex grit removal units each sized for 130 ML/d. The influent and effluent channels constructed for the 2031 expansion will need to be sized to accommodate the 2061 peak hour flow.

7.9.3 Side Stream Bypass Pipe

The expanded vortex grit facility will pass 220 ML/d (plant flows from 200 to 420 ML/d) to the side stream bypass pipe. The 2061 vortex grit facility will accommodate 480 ML/d (plant flows from 200 to 680 ML/d) and as such the side stream bypass pipe exiting the grit facility shall be design to accommodate 480 ML/d. It is proposed to locate this pipe outside of the footprint of the expanded grit facility. The proposed side stream bypass pipe will pass through the proposed dewatering / thickening building. It is anticipated that the pipe would be located in an accessible pipe gallery in the basement of the proposed dewatering / thickening building.

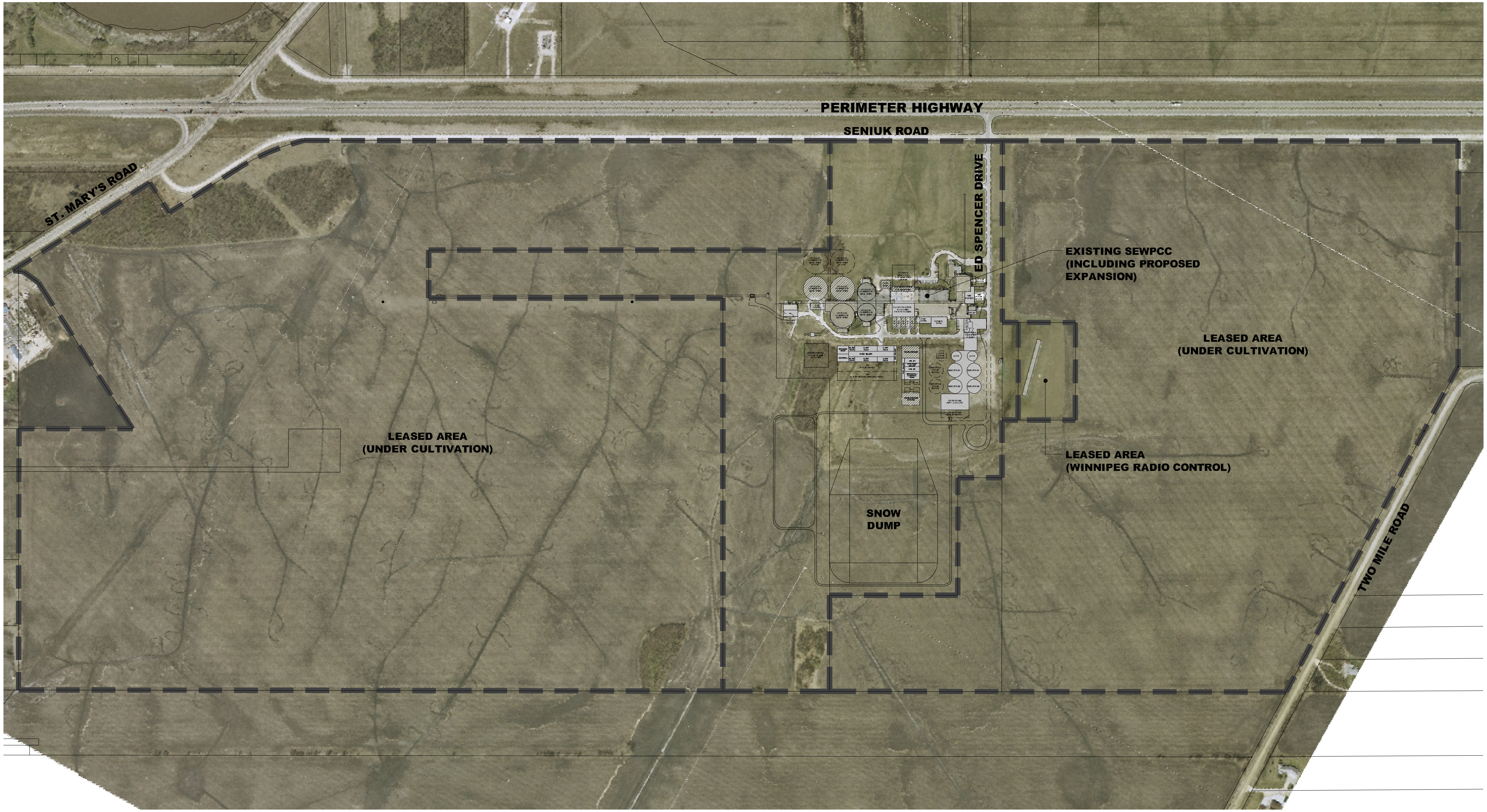
The proposed side stream bypass pipe on the south side of the site will be located between the existing plant and the proposed BAF / HRC facility to minimize the piping required, allow for efficient flow splitting between the main stream and the side stream processes, and to allow for unobstructed future expansion of the BAF / HRC facility to the south.

The proposed side stream bypass pipe will receive and split the following flows:

- An overflow located at the south end of the existing primary clarifier influent channel would direct 50 ML/d (plant flows 150 to 200 ML/d) to the side stream bypass pipe. This pipe will be sized to accommodate 200 ML/d so that all flows from the existing grit tanks could bypass primary clarification in an emergency situation. The side stream bypass pipe from this location forward will be sized to accommodate the 2061 peak hour flow of 680 ML/d.

- The side stream bypass pipe flow of 270 ML/d (plant flows from 150 ML/d to 420 ML/d) enters a splitter box upstream of the HRC. A flow of 175 ML/d (plant flows from 150 to 325 ML/d) is directed to the HRC for the 2031 design and 375 ML/d (plant flows from 150 to 525 ML/d) is directed to HRC for the 2061 design. Plant flows greater than 325 ML/d (2031 design) and 525 ML/d (2061 design) will flow over a weir to the bypass pipe and be conveyed to the river without further treatment.

- Flow from the HRC is split between the BAF via the intermediate pump station 50 ML/d (plant flows from 150 to 200 ML/d) for the 2031 design, 150 ML/d (plant flows from 150 to



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LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE — DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO.	REVISIONS	DATE	BY

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 Fax 204-453-9012

DESIGNED BY	J.B.	CHECKED BY	J..B
DRAWN BY	K.R.	APPROVED BY	S.K.B.
HOR. SCALE:	1:750	RELEASED FOR CONSTRUCTION:	
VERTICAL:	–		
DATE	2011.11.30	DATE	

ENGINEER'S SEAL

TENDER NO.

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

LAND AVAILABILITY

CITY DRAWING NUMBER
FIGURE 7.10

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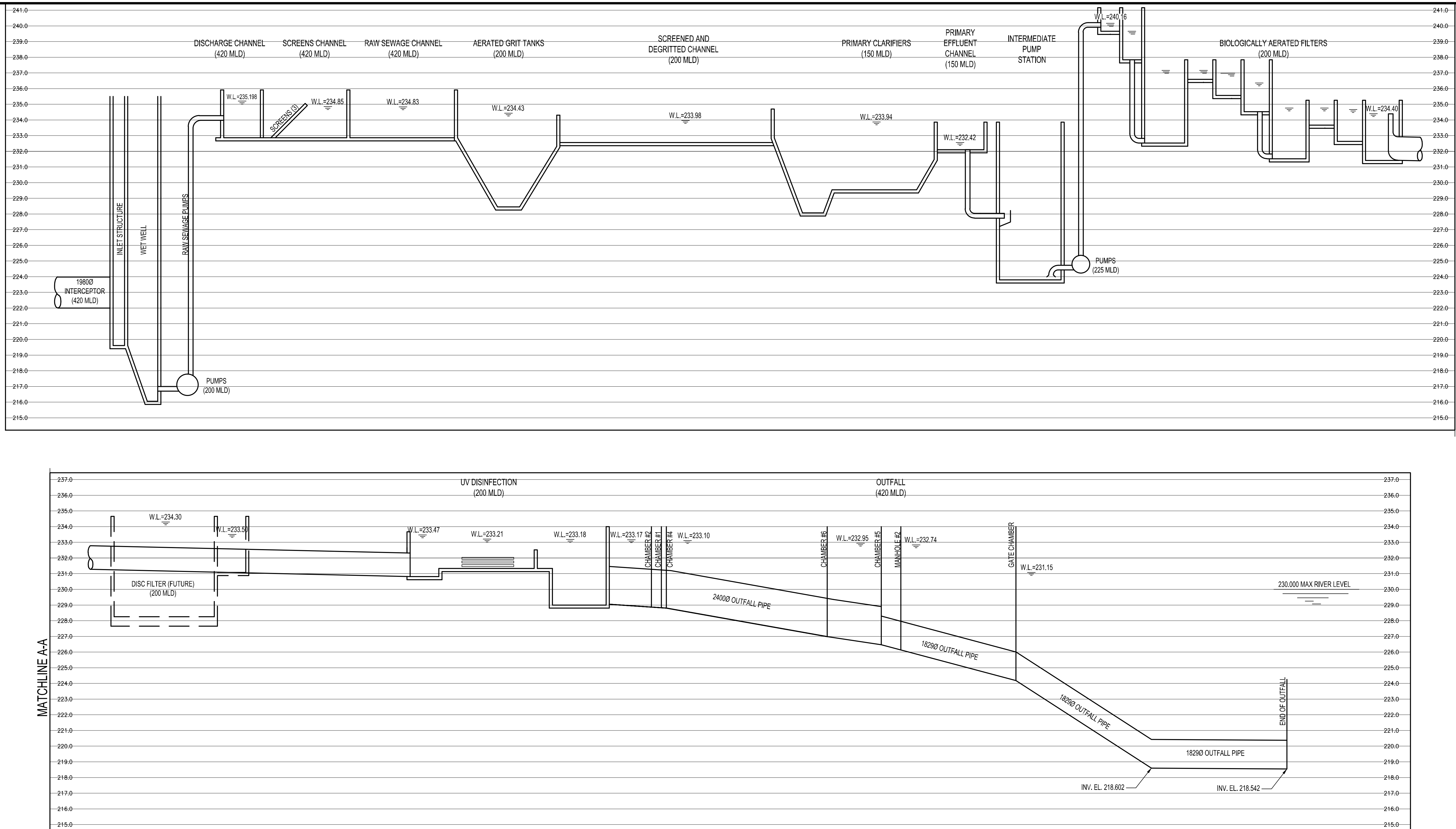

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UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE | DATE

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DRAWN BY	APPROVED BY
HOR. SCALE: N.T.S. VERTICAL: 1:250	RELEASED FOR CONSTRUCTION:
DATE 2008.11.03	DATE

ENGINEER'S SEAL

TENDER NO.

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

MAIN STREAM HYDRAULIC PROFILE @ 420 MLD FOR 2031 DESIGN

CITY DRAWING NUMBER: FIGURE 7.11

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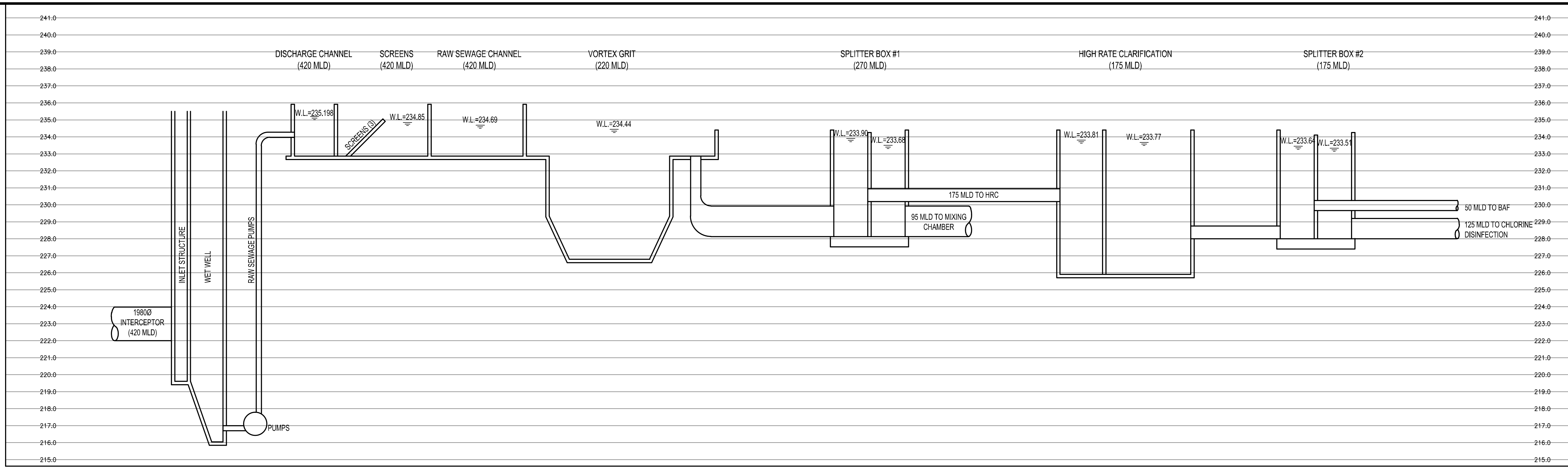

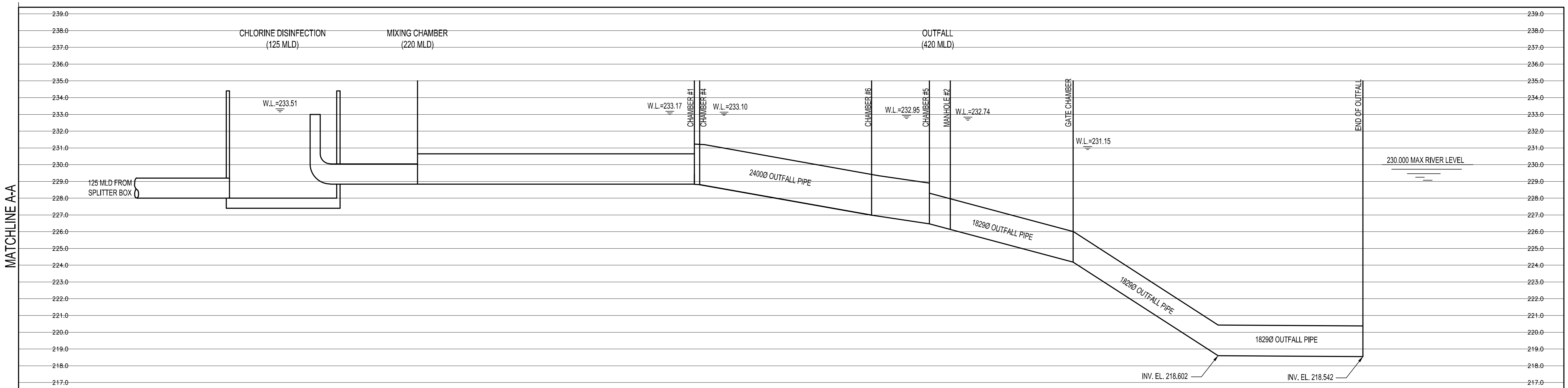

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE | DATE

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B.M. ELEV.

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DESIGNED BY | CHECKED BY
DRAWN BY | APPROVED BY
HOR. SCALE: N.T.S.
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RELEASED FOR CONSTRUCTION:
DATE 2008.11.03 | DATE

ENGINEER'S SEAL

TENDER NO.

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

SIDE STREAM HYDRAULIC PROFILE @ 420 MLD
FOR 2031 DESIGN

CITY DRAWING NUMBER
FIGURE 7.12

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300 ML/d) for the 2061 design and chlorine disinfection 125 ML/d (plant flows 200 to 325 ML/d) for the 2031 design, 225 ML/d (plant flows from 300 to 525 ML/d) for the 2061 design. Following chlorine disinfection, flow is discharged back to the side stream bypass pipe.

The proposed side stream bypass pipe connects back into the existing outfall downstream of the UV facility at existing Outfall Chamber No. 1.

7.9.4 Primary Clarification

The existing primary clarifiers will be converted to CEPT to reduce TSS and precipitate phosphorous. The primary clarifiers will receive flow up to 150 ML/d. A wet well for the intermediate pump station would be constructed adjacent the south end of the existing primary clarifier effluent channel. A pipe will hydraulically connect the channel to the wet well.

It is proposed to locate the CEPT chemical handling facility south of existing Primary Clarifier No. 3. This location will facilitate access for chemical delivery and would be reasonably close to the chemical injection location. Operator access to this facility will be through existing Primary Clarifier No. 3.

Primary sludge would be transferred to a balancing tank located in the new dewatering / thickening facility through the existing Gallery No. 3 and the existing boiler room.

7.9.5 High Rate Clarification

Two (2) HRCs are proposed as part of the 2031 design and it is anticipated that an additional two (2) HRCs will be required for the 2061 design. Flow from the HRC is split between the BAF via the intermediate pump station 50 ML/d (plant flows from 150 to 200 ML/d) for the 2031 design, 150 ML/d (plant flows from 150 to 300 ML/d) for the 2061 design and chlorine disinfection 125 ML/d (plant flows 200 to 325 ML/d) for the 2031 design, 225 ML/d (plant flows from 300 to 525 ML/d) for the 2061 design.

At this stage in the design the HRC vendor has not been selected. The footprint shown on the layout is for Degremont Desadag®, as it has the larger footprint. If Veolia's Actiflo™ is selected, the foot print will decrease. It is proposed to locate the HRC / BAF facility south of the existing south access road so that access to the sludge hauling facility is maintained throughout construction and to provide access to the existing service building and proposed CEPT chemical facility following construction.

Sludge from the HRC process will be pumped to a balancing tank in the proposed dewatering / thickening facility. The chemical room, mechanical room and electrical room required for the HRC will be located adjacent the HRC. This facility could also include a washroom, laboratory and office space.

7.9.6 Intermediate Pump Station

The pump station will include a wet well with an accessible pump gallery similar to the set up for the existing raw sewage pump station. The pump gallery will be accessible from existing

Gallery No. 5 and from Primary Clarifier No. 3. The 2031 design will have a pump capacity of 225 ML/d and will provide a future provision to accommodate the 2061 pumping requirement of 338 ML/d. The flow received is transferred to the influent channel of the proposed BAF.

The intermediate pump station will initially receive 150 ML/d (plant flow from 0 to 150 ML/d) from the existing primary clarifiers, 50 ML/d (plant flow from 150 to 200 ML/d) from the HRC process for the 2031 design, and 25 ML/d from the backwash clarifiers. In the future it will receive an additional 100 ML/d (plant flow from 200 to 300 ML/d) for the 2061 design and an additional 9 ML/d from the backwash clarifiers.

7.9.7 Biological Aerated Filter

The BAF proposed for the 2031 design has a capacity of 200 ML/d. The BAF receives 200 ML/d of primary effluent and 25 ML/d of clarified backwash waste and wastes 18 ML/d through backwashing. The BAF capacity required for the 2061 loads is 300 ML/d. The BAF capacity for 2061 was determined by extrapolating the loads for the 2031 population to the 2061 population. BAF space will be allotted to the south to accommodate the increased population growth.

At this stage in the design the BAF vendor has not been selected. The footprint shown is based on preliminary information provided by Degremont for the BIOFOR®, as it has the largest footprint. Based on preliminary information, if Veolia's Biostyr™ is selected the foot print will decrease. As mentioned previously, it is proposed to locate the HRC / BAF facility south of the existing south access road. This location would ensure access to the sludge hauling facility is maintained throughout construction and provide access to the existing service building and proposed CEPT chemical facility following construction. The electrical room and blower room for the BAF will be located at the west end of the BAF over the effluent storage (BIOFOR® only) and backwash waste storage chambers.

Backwash waste from the BAF (18 ML/d for 2031, 27 ML/d for 2061) will be pumped from the BAF backwash waste storage tanks to the west end of the existing MLSS channel for distribution to the backwash clarifiers. Effluent from the BAF (200 ML/d for 2031, 300 ML/d for 2061) will be discharged by gravity to the UV facility.

7.9.8 Backwash Clarification

The two (2) existing 33.5 m diameter secondary clarifiers will be reused to clarify the backwash waste flow (18 ML/d) for the proposed 2031 design. Backwash waste from the BAF is directed to the existing MLSS channel where it will be split between the two clarifiers. It is anticipated that chemicals will be required to facilitate clarification. The existing 45.7 m dia. clarifier will be required to clarify the backwash flow (27 ML/d) for the 2061 design. The settled sludge would be pumped through the existing gallery no. 3 and the existing boiler room to a balancing tank located in the proposed dewatering / thickening building.

7.9.9 UV Disinfection

The proposed UV disinfection facility will be constructed adjacent to the existing UV facility to accommodate 200 ML/d (2031 design) with the future provision to expand the facility to 300 ML/d (2061 design). All effluent from the BAF will be directed to the UV. The UV would be separated from the BAF to allow for tertiary filters to be installed in the future if further total phosphorus removal becomes a requirement.

7.9.10 Chlorination / Dechlorination

The existing HPO reactors will be reused to provide chlorine contact time. Existing Reactor No. 4 will be used as a buffer tank, while the existing Reactor No. 3 would be retrofit to provide chlorine contact time for the 2031 flow of 125 ML/d (plant flows from 200 to 325 ML/d). Existing HPO Reactor No. 2 will also be required along with existing HPO Reactor No. 3 to provide chlorine contact time for the 2061 flow of 225 ML/d (plant flows from 300 ML/d to 525 ML/d).

It is proposed to convert the existing HPO equipment room into the chemical room for chlorine disinfection and dechlorination. It has been assumed the onsite generation of sodium hypochlorite will be used for chlorine disinfection and sodium bisulfite will be used for dechlorination.

7.9.11 Outfall

The existing outfall was upgraded in 2010 to include an effluent monitoring facility and a portion of the outfall was twinned to provide additional capacity during high river events. The twinned portion of the outfall includes an 1829 mm dia. pipe in parallel with a new 2400 mm diameter pipe. The remaining portion of the outfall is 1829 mm diameter.

The proposed design for 2031 does not include any modifications to the outfall except for the tie-in of the side stream bypass pipe to Outfall Chamber No. 1. In order to pass the projected peak 2061 flow of 680 ML/d the outfall will require further twinning. It is proposed to complete the twinning of the outfall with a new 2400 mm diameter pipe installed up to the existing gate chamber at the river. It is also proposed to replace the existing 1800 mm dia. pipe from the gate chamber to the outlet with a new 3000 mm dia. pipe, as the existing pipe from the gate chamber to the outlet contains constrictions that induce high head losses at high flow. Preliminary modeling indicates that these upgrades will be sufficient to pass the 680 ML/d. Refer to Section 7.10 for further details regarding hydraulic modeling.

As noted in section 4.24 Peak Flows to the SEWPCC, the existing collection system hydraulic capacity cannot safely convey the existing wet weather flows to the SEWPCC. Emergency overflows occur when the hydraulic capacity of the collection system is exceeded to protect against wide-spread basement flooding. To convey more wet weather flows to the SEWPCC will require the existing capacity of the collection and conveyance system to be increased. The increase of flows to the SEWPCC has implications to the hydraulic design of the plant unit operations and processes. As part of the 2031 design the bypass piping will be designed to accommodate the 2061 flow of 680 ML/d. Prior to implementing any treatment plant expansions beyond the 2031 design horizon, Stantec recommends that a dedicated review be undertaken to identify options and possible future collection system upgrades/treatment options as compared to providing additional wet weather flow treatment at the SEWPCC. This review will better define the long-term South End service area collection/treatment strategy.

7.9.12 Biosolids

The proposed design calls for biosolids to be digested and dewatered onsite at the SEWPCC. The biosolids components assumed for the SEWPCC include:

- A sludge blend tank system to receive primary sludge, HRC sludge and backwash sludge. Sludge piping from the existing primary clarifiers and the backwash clarifiers will pass through existing Gallery No. 3 and travel through the existing boiler room before entering a new pipe gallery located along the entire west side of the proposed dewatering / thickening building. The sludge blend tank will be located below the thickening equipment. Pumps located in a basement gallery will be used to transfer the sludge from the blend tank to the thickening equipment.

- For this validation report, drum thickeners were assumed to thicken the blended sludge prior to digestion. Alternative thickening processes can be addressed during preliminary design. Two (2) duty units and one (1) standby unit have been assumed. Floor space has been allotted in the thickener room for additional thickening equipment required for the 2061 design.

- Two stage anaerobic digesters for sludge digestion. Centrifuge dewatering was chosen as it is a technology of choice by the City at the NEWPCC facility. Four (4) tanks are required for the 2031 design, while an additional two (2) tanks are required for the 2061 design. A pipe gallery and space for digester mixing equipment has been allotted between the digesters. The pipe gallery will be connected to the pipe gallery for the proposed dewatering / thickening building. Two (2) additional digesters required for the 2061 design would be constructed west of the proposed digester location.

- Two (2) sludge holding tanks (SHT) to store digested sludge before dewatering. Space has been allotted between the SHTs for piping, mixing equipment and pumps to transfer the digested sludge to the dewatering equipment. An additional SHT required for the 2061 design will be located west of the proposed SHT location.

- Centrifuges to dewater digested sludge. Centrifuge dewatering was chosen as it is a technology of choice by the City at the NEWPCC facility. Two (2) duty and one (1) standby centrifuge have been assumed. Floor space has been allotted for additional centrifuges required for the 2061 design. The dewatering setup assumed is similar to that for the NEWPCC. The centrifuges would be located on an intermediate level with dewatered sludge pumps located below the centrifuges. The dewatered sludge pumps will transfer the dewatered sludge to bins located above the truck bay. Two lanes have been allotted for sludge hauling trucks.

- Space in the dewatering / thickening building for an electrical room, control room, mechanical room, washroom, and chemical room.

7.9.13 Future Effluent Requirements

Space has been allotted on the site to account for more stringent total nitrogen and phosphorous limits. A future nitrogen limit of 10 mg/L and a future total phosphorous limit of 0.3 mg/L have been assumed.

Additional BAF nitrification and denitrification cells will be required to further reduce total nitrogen. While the BAF required for the future nitrogen limit has not been sized, a generous space allocation for additional BAF denitrification cells has been provided south of the proposed 2031 BAF location. The new cells would operate in parallel with the existing BAF to provide the additional surface area required for additional denitrification.

To achieve a future effluent total phosphorous limit of less than 0.3 mg/L tertiary filtration would be required. Disc filtration has been assumed for the purpose of this analysis. The effluent from the BAF will pass through the filters before being directed to the UV disinfection facility. A headloss of 0.8 m was assumed for the disc filters and has been built into the proposal hydraulic profile and therefore it has been assumed additional intermediate pumping will not be required.

7.9.14 Future Biological Phosphorous Removal

The Program Team indicated that Manitoba Conservation may require biological phosphorous removal in the future. One option for biological phosphorous removal with the proposed BAF design is to remove carbon and phosphorous biologically with a Phoredox type process (expanding and reutilizing the existing HPO system) upstream, followed by nitrification / denitrification using the BAF. To accommodate future biological phosphorous removal with the proposed design, the following modifications are proposed:

- Add anaerobic cells upstream of the existing HPO reactors and add an additional HPO bioreactor. The size required for the anaerobic cells and additional HPO reactor has been estimated by scaling up the 175 MLD design developed as “Option K” through the SEWPCC Upgrading / Expansion Preliminary Design Report (Stantec, 2008) to 200 MLD. The anaerobic cells will be constructed south of the existing HPO reactors, while the new HPO reactor and any future HPO reactors required to accommodate the 2061 design will be constructed north of the existing HPO reactors. Space to house additional equipment (or conversion from pressure swing adsorption (PSA) to vacuum swing adsorption (VSA)) required to support the additional HPO reactor has been allocated north of the existing HPO equipment room. Alternatively, it is possible to implement an air-based system and size the facilities accordingly. As there are no conflicts with the implementation of an air-based system north of the existing facilities, the actual footprint can be assessed as part of future process designs and engineering design phases.

- The primary effluent from the HRC that is directed back to the BAF via the pump station will be piped to the anaerobic cells. It is assumed that this is hydraulically feasible, although would need to be confirmed through subsequent design stages.

- Convert the backwash clarifiers back to secondary clarifiers and add two additional 45.7 m diameter secondary clarifiers to provide an overall secondary clarification capacity of 200 MLD. The existing mixed liquor suspended solids (MLSS) channel will be expanded to the north and the new secondary clarifiers and future secondary clarifiers will be installed east and west of the extended MLSS channel.

- Convert the C-N-DN BAF to an N-DN BAF and pipe the backwash waste back to the front end of the primary clarifiers.

- Revise the intermediate pump station to pump secondary effluent to the BAF. The pump station could be constructed either south and adjacent to the primary clarifiers or at the west end of the secondary clarifiers. Both locations are shown on the site plan. If located south and adjacent to the primary clarifiers, a pipe connecting into the effluent channel downstream of the secondary clarifiers is required and the intermediate pump station would need to be constructed at a lower elevation than described in Section 7.3. If the pump station is located west of the secondary clarifiers, longer piping runs to the BAF and HRC are required and the pump station would need to be constructed at a lower elevation than described in Section 7.3. While both options are feasible, further analysis is required during subsequent design phases to determine the most appropriate location.

- The current site plan assumes chlorination / dechlorination for wet weather flows. The existing HPO reactors would no longer be available for chlorination / dechlorination and therefore a new facility would need to be constructed south of the proposed HRC facility. The chlorination / dechlorination feed and storage equipment would be located on top of the chlorine contact tank.

7.9.15 Geotechnical Investigations

A considerable amount of geotechnical information for the SEWPCC site exists and is available for use in the upgrade and expansion of the SEWPCC. Data collected from past geotechnical investigations and construction activities indicate that the site is fairly consistent in nature and soil composition, and well suited for the type of facilities being proposed. Given the clay conditions found on site and the type of major facilities to be added or expanded upon, piles will be required to prevent movement that could damage these structures. It is anticipated that dewatering will be required to facilitate safe and reliable foundation construction and piling.

Due to scheduling conflicts and seasonal constraints, it was not possible to conduct geotechnical investigations as part of this assignment. Nonetheless, Stantec reaffirms that it is prudent and good practice to undertake a specific and targeted geotechnical investigation program to mitigate potential risks associated with soil movement that could negatively impact the structural integrity of new and expanded facilities associated with the upgrade and expansion of the SEWPCC. As such, it is recommended that test holes be drilled in the proposed location of the HRC, BAF, UV disinfection facility, intermediate pumping station, and two additional holes in the general expansion area. The proposed geotechnical investigations is to take place as part of the schematic design phase and use information contained in this report (i.e., site layout diagrams) to develop a targeted investigation campaign. Soil samples will be collected and analyzed in a laboratory to properly characterize the composition and their mechanical properties. This information will be used in the structural design of the facilities to address risks associated with the geotechnical stability of existing and new facilities, and conveyance systems. The geotechnical investigations will be used to determine soil conditions at specific locations for the proposed structures, and recommend foundation designs, as well as any other special construction requirements.

7.10 HYDRAULIC PROFILE AND FLOOD PROTECTION

This section discusses the hydraulic profile for the proposed design and the level of flood protection for the facility.

7.10.1 Hydraulic Model

The InfoWork™ model developed for the SEWPCC Flood Protection Measures (Stantec, 2011) project was modified as required to reflect the proposed 2031 and 2061 designs. The model was used to determine the hydraulic profile for the proposed SEWPCC design by modeling peak flows at various river levels.

7.10.2 Red River Levels

A major factor influencing the SEWPCC hydraulic profile is the level of the Red River at the SEWPCC outfall. Table 7.13 indicates the Red River levels at the SEWPCC outfall for various spring return periods. The data presented in Table 7.13 is based on the recently expanded floodway. The highest river level experienced at the SEWPCC outfall was 231.444 (Cochrane Engineering Inc., 1997) during the 1997 flood.

Table 7.13 - Red River Elevations at the SEWPCC Outfall for Various Return Periods

Return Period	River Level at SEWPCC Outfall (m)
5	228.1
10	228.4
20	228.4
33	228.7
50	229.8
100+	231.2

Source: Grant Mohr, City of Winnipeg, Water and Waste Dept., July 17, 2006

7.10.3 Hydraulic Profile

The hydraulic plant profile consists of two parts, the main stream hydraulic profile and the side stream hydraulic profile. Refer to Figures 7.11 and 7.12 for hydraulic profiles for the proposed 2031 design at the total pumped flow of 420 ML/d. Refer to Appendix A for modeling outputs for the peak day flow and total pumped flow related to the 2031 and 2061 proposed designs. Refer to the Figure 6.2 for the process flow schematic developed for the 2031 design and to Figure 7.13 for the process flow schematic developed for the 2061 design for flow splitting between the main stream and side stream.

7.10.4 Main Stream Hydraulic Profile

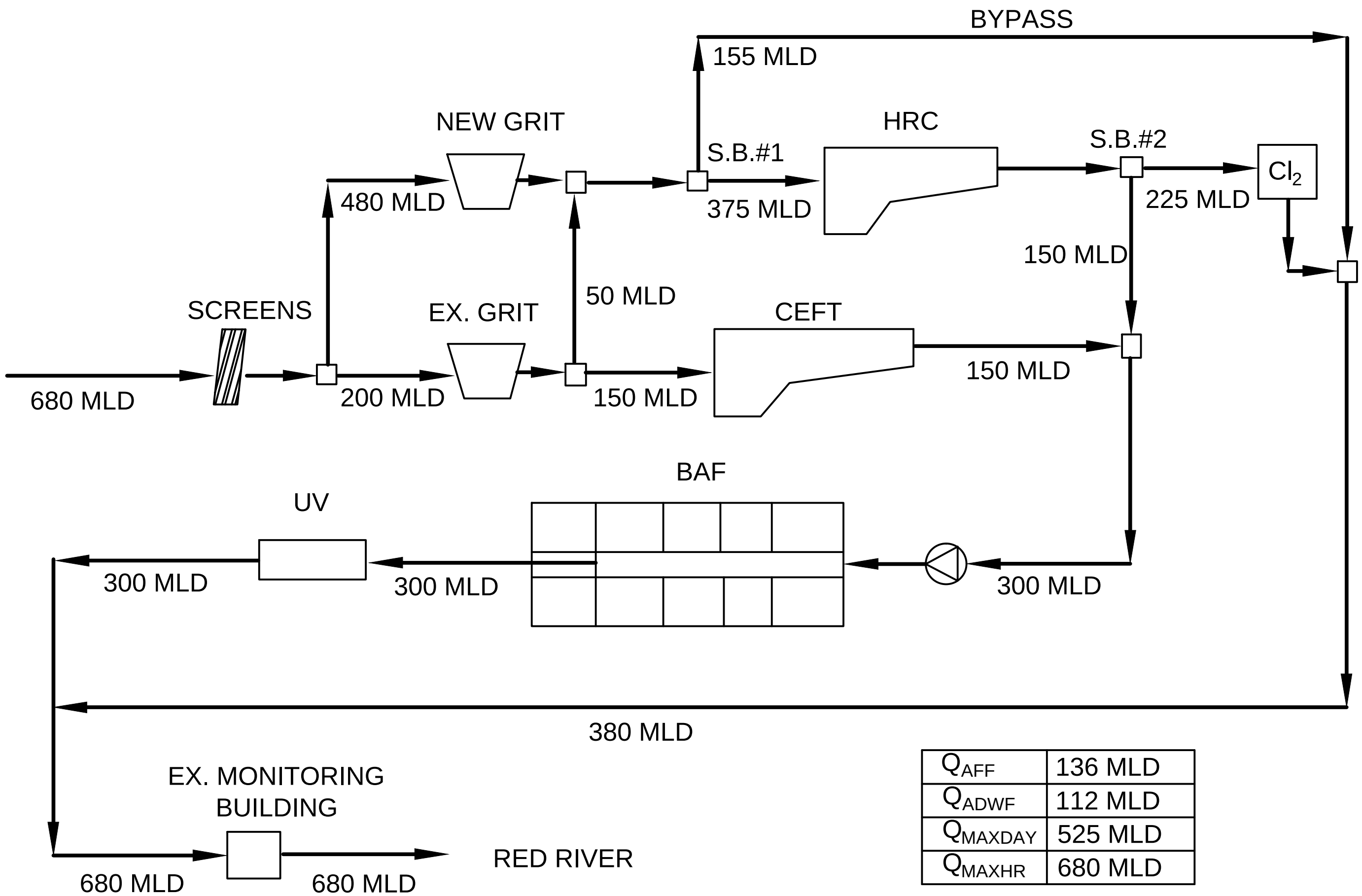
The proposed main stream process includes the existing raw wastewater pump station, screens, grit and primary clarifiers. Following the primary clarifiers, flow is pumped from the proposed intermediate pump station to the inlet of the BAF, from where it flows by gravity through UV disinfection and outfall to the Red River. Each unit process has the capacity as listed in Table 7.14. Flow in excess of the capacity noted is diverted to the side stream.

Table 7.14 - Proposed Capacity for Main Stream Processes

Unit Process	2031 Capacity	2061 Capacity
Raw Wastewater Pumping	420 ML/d	680 ML/d
Screens	420 ML/d	680 ML/d
Aerated Grit Tanks	200 ML/d	200 ML/d
Primary Clarifiers	150 ML/d	150 ML/d
Intermediate Pump Station	225 ML/d	338 ML/d
BAF	200 ML/d	300 ML/d
UV Disinfection	200 ML/d	300 ML/d
Outfall	420 ML/d	680 ML/d

The existing UV system was originally designed to only disinfect dry weather flows up to 100 ML/d when the river level was less than 229.00 at the SEWPCC outfall. The UV facility was taken out of service on a number of occasions over the past 15 years, as the river level has been greater than 229.00, both before and after the completion of the floodway expansion. The new licence for the SEWPCC requires disinfection of maximum day flows (325 ML/d) at all river levels.

The first step in determining the hydraulic profile for the main stream is to determine at what river level the existing UV facility can provide disinfection when overall plant flows are 325 ML/d. The model was run with 200 ML/d through the existing UV and 125 ML/d through the side stream bypass and it was determined that with the existing UV facility channel elevations, disinfection would no longer be effective at a river level of 228.65 m. In order to meet the Licence either the outfall needs to be expanded or the UV channel needs to be reconstructed at



Q_{AFF}	136 MLD
Q_{ADWF}	112 MLD
Q_{MAXDAY}	525 MLD
Q_{MAXHR}	680 MLD

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THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

PRELIMINARY FLOW SCHEMATIC
FOR 2061

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FIGURE 7.13

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a higher elevation. Stantec has assumed the UV reconstruction option as it is the least costly option and the UV channel will require expansion to accommodate the increased flows.

The next step in determining the hydraulic profile for the main stream is to select a design river elevation for disinfection. Stantec has assumed a design river level of 230.00, as it exceeds the 50 year return period for the SEWPCC. The model was run with 200 ML/d exiting the UV facility and 125 ML/d passing though the side stream bypass and it was determined that UV disinfection is effective at all times with the water level downstream of UV disinfection at 231.99 m. This equates to increasing the UV channel elevation by 1.66 meters. The model was then run with the UV channel elevation raised by 1.66 meters and the total pumped flow of 420 ML/d (200 ML/d through the UV and 220 ML/d though the side stream bypass) and it was determined that while the disinfection effectiveness would be reduced, flooding of the UV facility would not occur. Modeling also determined that the maximum river level where UV disinfection would be effective under the total pumped flow of 420 ML/d is 228.72 m (33 yr. return period).The elevation of the UV disinfection facility could be lowered by 0.7 m if 410 m of the remaining single 1800 mm diameter outfall were twinned with 2400 mm diameter pipe. Lowering the UV facility would reduce the hydraulic grade line through the BAF, thereby lowering the intermediate pumping requirements and associated energy consumption costs. This was not proposed because the energy savings were estimated to be less than $10,000 per year in comparison with a capital cost of approximately $2,000,000 for twinning the outfall.

Flooding of floors will not occur in any facility if the river level is less than 230.00. Refer to Table 7.15 for minimum level that electrical equipment can be located at various facilities. If the river level is greater than 230.00 flow shedding in the collection system is required to prevent flooding at the SEWPCC. Flow shedding was undertaken during the 2011 flood season and was documented in the document SEWPCC Flood Protection Measures (Stantec, 2011). Additional modeling is required during subsequent design phases to determine the amount of shedding required at various river levels.

Table 7.15 – Proposed "Out of Water" Elevation for Design River Level (230.00)

Unit Process	Elevation (m)
Headworks	225.915
CEPT	234.50
HRC	234.50
Backwash Clarification	232.766
UV Disinfection	234.530

7.10.5 Side Stream Hydraulic Profile

The side stream process includes the existing raw wastewater pumping, screens, proposed vortex grit removal units, splitter box, high rate clarifiers, another splitter box and chlorine disinfection before the side stream bypass pipe connects into the existing outfall downstream of UV disinfection at Outfall Chamber No. 1. Flow at the first splitter box is diverted to the high rate clarifier with the additional flow bypassing the remaining processes to the outfall. The second splitter box diverts flow between the intermediate pump station and chlorine disinfection. Refer to Table 7.16 for the proposed capacity of each side stream process.

Table 7.16 - Proposed Capacity for Side Stream Processes

Unit Process	2031 Capacity	2061 Capacity
Raw Wastewater Pumping	420 ML/d	680 ML/d
Screens	420 ML/d	680 ML/d
Vortex Grit Tanks	220 ML/d	480 ML/d
Splitter Box No. 1	270 ML/d	530 ML/d
High Rate Clarification	175 ML/d	375 ML/d
Splitter Box No. 2	175 ML/d	375 ML/d
Chlorine Disinfection	125 ML/d	225 ML/d
Side Stream Bypass	220 ML/d	380 ML/d
Outfall	420 ML/d	680 ML/d

The hydraulic profile for the side stream is determined by the existing screen channel and the headloss through the proposed processes. At the proposed design river elevation of 230.00 m and the peak pumped flow of 420 ML/d the proposed vortex grit units, high rate clarifiers and chlorine disinfection contact tank fit within the hydraulic profile. The side stream was modeled at a plant flow of 420 ML/d and flooding did not occurred in the high rate clarifiers until the Red River reaches an elevation of 231.00 m.

7.10.6 50-Year Design

As noted in Section 7.9.11, the outfall requires expansion to meet the total pumping capacity of 680 ML/d that is projected for the year 2061. The model was modified to account for this as follows:

- Twin outfall by extending 2400 mm dia. pipe from existing outfall chamber no. 6 to the gate chamber at the river.
- New 3000 mm dia. pipe from the gate chamber into the Red River.

The hydraulic model was revised to include the outfall modification noted above and it was confirmed that the projected future total pump capacity of 680 ML/d could be passed to the river without resulting in flooding at the UV facility. If the outfall modifications were undertaken as part of the 2031 design, the proposed elevation for the UV disinfection facility would only need to be raised 0.845 m (instead of 1.66 m) to ensure disinfection at the projected future maximum day flow of 525 ML/d. For the purpose of this report it has been assumed that the outfall will not be twinned for the 2031 design and the UV disinfection facility will be raised 1.66 m.

7.11 ADDITIONAL ITEMS REQUIRING FURTHER INVESTIGATION IN SUBSEQUENT DESIGN STAGES

This section has identified the major components and their interrelation to validate the design proposed for the SEWPCC. Other aspects that have not been investigated at this stage and will require further investigation in subsequent design stages are summarized in Table 7.17. These components were not deemed to be relevant to validating the design concept and therefore were not investigated.

Table 7.17 – Additional Items Requiring Investigation at Subsequent Design Stage

Item	Action
Siteworks	• Determine site fencing requirements • Determine site fire protection requirements
Administration Building	• Determine scope of Administration Building upgrades
Asbestos Abatement	• Confirm scope of asbestos abatement
Architectural	• Determine new building construction and exterior finish
Process	• Determine the flushing water requirements for new equipment and determine modification required for to the existing flushing water system
Electrical	• Evaluate electrical loads for the upgrade and determine Manitoba Hydro power requirements • Determine transformer requirement and determine if transformers will be pre-purchased due to long lead time for delivery • Determine emergency power requirements • Determine security system requirements
Controls	• Determine process for conversion to a PLC based control system and SCADA system • Determine if PLC hardware will be pre-selected • Determine if SCADA system will be pre-selected
Odor Control	• Determine the odor control requirements

8.0 Civil Asset Condition Assessment

8.1 INTRODUCTION

The objective of this task was to:

- Gather available information on the plant condition from previous assessment reports
- Identify items that have been addressed since the last condition assessment
- Identify information gaps
- Develop an assessment methodology for the HPO Reactors.

8.2 PREVIOUS WORK AND SCOPE

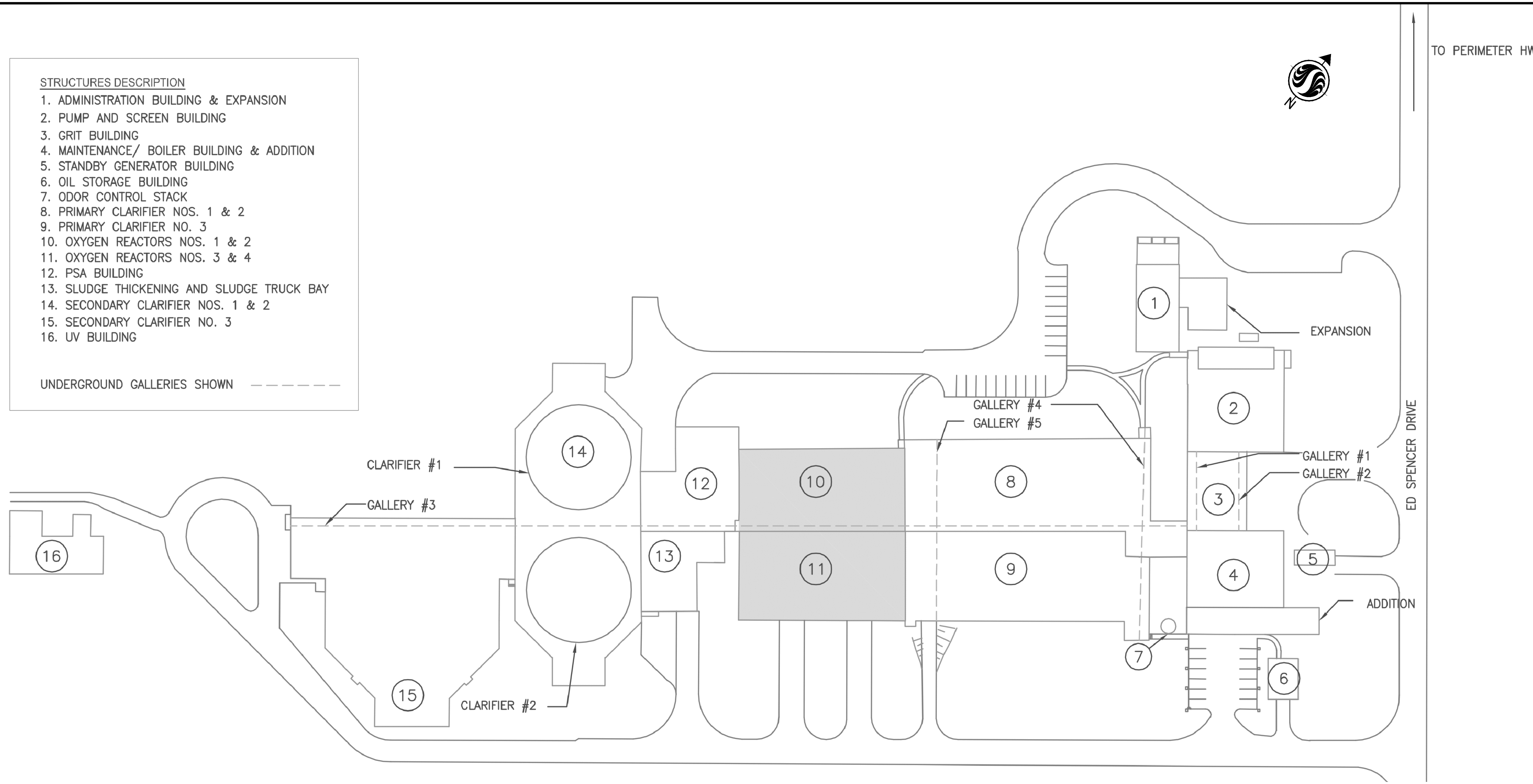
Stantec conducted a General Facility Condition Assessment in the spring of 2008 of the South End Water Pollution Control Centre located on Lot #149, St. Mary's Road in South Winnipeg. The assessment was limited to providing a general, non-destructive, walk-through review of the structural and building envelope systems only.

On February 29, 2008, Vector Corrosion Technologies was contracted to conduct testing on the concrete of Primary Clarifier #3 at the South End Water Pollution Control Centre. Subsequently on March 12, 2008, Vector Corrosion Technologies was contracted to test the concrete of Primary Clarifier #1. The purpose of this invasive investigation was to assess the concrete condition and determine to what extent, if any, are issues regarding reinforcing steel corrosion associated with the Primary Clarifiers.

The general purpose of the Condition Assessment was to provide information needed to evaluate the current structural and building envelope conditions of the facility, and identify potential problem areas.

The 2008 condition assessment of the facility was based upon:

- A review of available construction drawings of buildings and structures contained within the facility
- Informal discussions with plant staff regarding maintenance history and building performance
- A visual, non-destructive walk-through review of the facility
- Invasive testing of the Primary Clarifiers by Vector Corrosion Technologies



STRUCTURES DESCRIPTION

1. ADMINISTRATION BUILDING & EXPANSION
2. PUMP AND SCREEN BUILDING
3. GRIT BUILDING
4. MAINTENANCE/ BOILER BUILDING & ADDITION
5. STANDBY GENERATOR BUILDING
6. OIL STORAGE BUILDING
7. ODOR CONTROL STACK
8. PRIMARY CLARIFIER NOS. 1 & 2
9. PRIMARY CLARIFIER NO. 3
10. OXYGEN REACTORS NOS. 1 & 2
11. OXYGEN REACTORS NOS. 3 & 4
12. PSA BUILDING
13. SLUDGE THICKENING AND SLUDGE TRUCK BAY
14. SECONDARY CLARIFIER NOS. 1 & 2
15. SECONDARY CLARIFIER NO. 3
16. UV BUILDING

UNDERGROUND GALLERIES SHOWN - - - - - -

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE　DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO.	REVISIONS	DATE	BY

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 Fax 204-453-9012

DESIGNED BY　CHECKED BY
DRAWN BY　APPROVED BY
HOR. SCALE: N.T.S.
VERTICAL:　RELEASED FOR CONSTRUCTION:
DATE 2008.112.12　DATE

ENGINEER'S SEAL

TENDER NO.

Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

SITE PLAN

CITY DRAWING NUMBER　FIGURE 8.1　SHEET　OF

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2011 December 12, 11:17 a.m.

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Structural components undergoing review were examined with focus on the condition and performance of the primary structural systems. Deficiencies deemed insignificant to the structural integrity of the structures were not reported. The Building Envelope review addresses the condition and performance of the building envelope systems required to protect the structure and its interior components from damage due to moisture infiltration, air infiltration, and premature deterioration of building components.

The assessment is based, in part, on information provided by others. Unless specifically noted, Stantec assumed that this information is correct and have relied on it in developing our conclusions.

It is possible that unexpected conditions may be encountered at the facility, that were not identified in the 2008 investigation or issues that have arisen since this investigation. Should such an event occur, Stantec should be notified in order that we may determine if modifications to our conclusions are necessary.

8.3 2008 FACILITY ASSESSMENT

The SEWPCC consists of a number of structures constructed at various times between 1971 and 1998. For the purposes of this report, we have divided the SEWPCC facility into the divisions noted below, with the approximate dates of construction in parentheses. A site plan indicating location of the building components is included in Figure 8.1.

- Administration Building (1971) and Expansion (1990).
- Pump and Screen Building (1971).
- Grit Building (1971).
- Maintenance/Boiler Building (1971) and Addition (1991).
- Standby Generator Building (1991).
- Odor Control Stack (1988).
- Primary Clarifiers Nos. 1 and 2 (1971).
- Primary Clarifier No. 3 (1989).
- Oxygen Reactors Nos. 1 and 2 (1971).
- Oxygen Reactors Nos. 3 and 4 (1990).
- PSA Building (1971).
- Sludge Thickening and Sludge Truck Bay (1971).

- Secondary Clarifiers Nos. 1 and 2 (1971).
- Secondary Clarifier No. 3 (1990).
- UV Building (1998).

Each section summarize the findings of the 2008 condition assessment in bullet form. Through correspondence with Ron Hahlweg, Head Plant Operator, items addressed/repaired/fixed following the 2008 condition assessment were identified. Such items are discussed in each section providing an update to the 2008 condition assessment findings.

Photos identifying some of the issues noted are included in Appendix B.

8.3.1 General Comments

Hairline shrinkage cracks in concrete slabs and walls were noted throughout the facility. Signs of moisture migration through the cracks are not yet evident. The cracks should be monitored for signs of continued propagation as part of a regular building maintenance program.

During the 2008 condition assessment, mould was identified on the brick veneer at various locations. See Photo 1.3-1 and 1.3-20 as examples. Stantec was informed that pressure washing was implemented in 2009 to remove much of the mould but further washing is still required. Work Order No. 1004092 was issued in May 2010 to undertake additional cleaning and has not yet been completed. No preventative action to inhibit future mould growth has been implemented. The brick veneer should be cleaned and a sealant applied to prevent high moisture conditions and premature brick failure.

Exterior caulking at the windows requires replacement at most windows. Caulking of the flashing at the parapet corners requires servicing.

The photos referred to in Appendix A were taken in the Spring of 2008. Notes have been added identifying the remedial work that has been undertaken since the 2008 assessment.

8.3.2 Administration Building and Expansion

- The majority of the primary structural framing members are concealed by wall finishes and ceiling finishes. Reviews of the main floor and roof structure were made at select locations by removing ceiling panels.
- No signs of significant structural distress were noted at the time of the review.
- Minor hairline shrinkage cracks and spalling of concrete was noted at the exterior north entry slab in 2008. See Photo 1.3-2. This was patched in June 2009 to minimize further cracking.
- Plant staff has reported water infiltration into the northwest corner of the men's locker room located in the basement. A basement wall expansion joint is located near this corner and

may be the source of the leak. Investigation of this joint is recommended to confirm water entry point. The risk associated with allowing this leak to continue is deteriorated building components that will be more costly to correct in future.

- Moisture infiltration was noted at the interior wall construction joint at the ramp to the Grit Building and should be further investigated. The risk associated with allowing this leak to continue is deteriorated building components that will be more costly to fix in future.
- Mould growth was noted on the north wall of the Laboratory Storage Room. See Photo 1.3-3. This was cleaned and the wall was repainted in 2009. However, action to prevent future mould growth was not undertaken.
- Caulking at the corner parapet flashings should be replaced. See Photo 1.3-4. This has since been repaired and the roof leak has been fixed.

8.3.3 Pump and Screen Building

- The concrete ring wall, as viewed from the dry well, has experienced external water infiltration. The wall is wet in numerous locations and significant efflorescence has taken place. Areas of discoloration due to reinforcing steel corrosion as well as areas of exposed reinforcing steel are evident. See Photos 1.3-5, 1.3-6, 1.3-7, 1.3-8 and 1.3-9. This should be addressed immediately. Further investigation of the perimeter ring wall is recommended in order to assess the integrity of the concrete and reinforcing steel, and to establish measures to protect/reinforce the wall in order to improve its long-term performance. A program of concrete and reinforcement sampling and analysis will be required. As the wastewater lift station is one of the highest-valued pieces of infrastructure, maintaining the structural integrity should be a high priority. The risk associated with doing nothing is failure of the lift station wall and flooding of the station. We view this investigation as critical.
- The exterior entry slab linking the Pump and Screen Building to the Administration Building has exposed reinforcing steel and requires repair. See Photo 1.3-10.
- The west concrete stair has exposed reinforcing steel and requires cosmetic repair.
- The concrete on the wet well walls appear to be damaged. Spalled concrete requires repair.
- Water leaks at the hatch in the elevator machine room require investigation. Water leakage into the elevator could cause damage to the elevator equipment and should be resolved.

8.3.4 Grit Building

- Hairline cracks in concrete slabs with associated efflorescence stains are evident in the walkways around the grit tanks. The cracks appear to be shrinkage cracks and should be sealed using an epoxy injection system. The concrete slabs should also be tested to determine if the apparent moisture migration has affected structural integrity. A program of concrete and reinforcement sampling and analysis will be required. See Photo 1.3-11 and

1.3-12. Additional investigative and remedial work is required. The risk of doing nothing is continued deterioration and future structural problems.

- Concrete around the clarifier drain is spalled and requires repair. See Photo 1.3-13.
- Exposed reinforcing steel in the concrete stair in the Grit Area stair was noted and should be repaired. See Photo 1.3-14.
- The floor slab in the Grit Bin area is not draining properly and has standing water. This appears to be an inherent design issue. Attempts to improve drainage have been made with limited success. Further investigation to determine the required remedial work is required.
- Damaged flashing should be replaced. See Photo 1.3-15. The roofing and flashing was replaced in its entirety in 2008 under bid opportunity no. 428-2008.
- A roof leak was suspected in the southeast corner of the Grit Tank Area and was investigated. See Photo 1.3-16. The roofing and flashing was replaced in its entirety in 2008 under bid opportunity no. 428-2008.
- The skylight is cracked and requires replacement. See Photo 1.3-17. The skylight was replaced in 2008 under bid opportunity no. 428-2008.

8.3.5 Maintenance/Service Building

- Hairline stress cracks were noted on the workshop floor area over a basement column. See Photo 1.3-18. These cracks appeared old in nature and currently would not be considered structurally significant. Sealing of the cracks would be recommended to prevent wash-water from penetrating the slab.
- Cracks in masonry partition walls in the storage/tool room suggest that the main floor slab may have undergone deflection. Future monitoring of the main floor slab for signs of continued concrete block cracking is recommended. See Photo 1.3-19.
- The parapet flashing is rusted in places and should be scheduled for replacement.
- The door and frame to the southeast exit stair has shifted and is jamming. The door and frame must be replaced. The reason for shifting of the wall is not apparent and should be investigated.

8.3.6 Standby Generator Building

- The structure generally appears to be in good condition with no signs of significant structural distress.
- The stair in the link to the maintenance building is experiencing leaking at a concrete construction joint and should be investigated and repaired.

8.3.7 Odor Control Stack

- The operation of the stack prevented access into the below grade concrete structure. An external review of the structure did not reveal signs of significant structural distress.

8.3.8 Primary Clarifiers Nos. 1 and 2

8.3.8.1 Tanks Full Visual Inspection

- Hairline concrete cracks have developed along the clarifier walls as seen from the galleries. The crack patterns have both a vertical and horizontal orientation. The horizontal cracks suggest cracking due to bending stresses. The vertical cracks may be attributed to concrete shrinkage. Signs of moisture migration could not be detected. These cracks should be monitored for signs of leakage as part of an on-going maintenance program.

- A double tee roof section located in the fan room is damaged at bearing and requires repair.

- Spalling of concrete at the guardrails around the clarifier tanks should be repaired.

- Exposed reinforcing steel at the base of the tank wall is visible in Gallery No. 4 (north). This condition, and similar conditions, should be investigated to confirm the integrity of the concrete wall, and the areas of exposed reinforcing steel should be repaired. See Photo 1.3-21. Grouting and sealing of spalled concrete and exposed reinforcing was completed as part of Work Order 1005511, issued in 2010.

8.3.8.2 Primary Clarifier No. 1 Tank Empty Visual Inspection

- The floor of the tank did not appear to be coated; however, the wall coating system was easily removed with a hand scraper and is beyond its service life.

- In general, the exposed concrete and wear plate surfaces appeared to be in sound condition. Minor abrasion/wear of the concrete topping was noted leaving some aggregates exposed. The wear plates exhibited minor surface rusting and were generally covered with a light rust residue. The random hammer “soundings” of the concrete did not reveal any areas of concern. Standing liquid was found on the west side of the clarifier tank floor expansion joint, which could be due to a previous repair, or slight differential movement of the slab on either side of the joint.

- Analyses of the core samples indicate that potential for corrosion of the reinforcement is low and the corrosion activity to date has not degraded the concrete. Tests of the core samples suggest a compression strength range between 62.5 MPa and 92.5 MPa with average compression strength of 73.0 MPa. The concrete cover above the reinforcement bars varies between 38 mm to 64 mm for the walls, and is 150mm for the floor slab.

- As per ACI 222R-01, if the potential difference between the reinforcing bars is less than one (1) mV, then the reinforcing steel is deemed electrically continuous. Electrical continuity tested between the selected locations was deemed to be continuous.
- Five (5) concrete cores were extracted and tested for depth of carbonation of the concrete. All five (5) cores were only slightly carbonated, between 1/16" to 1/4".
- Chloride ion testing was done on cores removed at each test location throughout the clarifier. From each core three (3) slices were removed at defined intervals: half inch, 1 ½ inches and 2 ½ inches. Only three (3) of the fifteen (15) samples tested revealed chloride ions higher than the NACE recommended limit.

8.3.9 Primary Clarifier No. 3

8.3.9.1 Tank Full Visual Inspection

- Hairline concrete cracks have developed along the clarifier walls as seen from the gallery. The crack patterns have both a vertical and horizontal orientation. The horizontal cracks suggest cracking due to bending stresses. The vertical cracks may be attributed to concrete shrinkage. Signs of moisture migration could not be detected. These cracks should be monitored for leakage as part of an on-going maintenance program.
- A section of exterior concrete paving slab requires repair at the southeast corner of the structure. See Photo 1.3-22.

8.3.9.2 Primary Clarifier No. 3 Tank Empty Visual Inspection

- The floors and walls of the main tank did not appear to be coated. The floors and walls are covered with a light dusting of residue which is easily removed with a scrub brush.
- In general, the exposed concrete and steel rail surfaces appear to be in sound condition. Minor abrasion/wear of the concrete topping was noted leaving some fiber-mesh particles exposed. The rails exhibited minor surface rusting and are generally covered with a light rust residue.
- A build-up of white residue is evident on the scum collector arm. Surface pitting was also evident on the scum collector.

8.3.10 Oxygen Reactors Nos. 1 and 2

- The exterior rooftop paving slab, in general, exhibits thermal cracking in numerous areas. The slab is nonstructural but will require cracks to be sealed in order to prevent further deterioration. Some vegetation growth in these cracks exists. See Photo 1.3-24.

- Exposed reinforcing steel along the tank wall is visible in Gallery No. 5 (north). This condition, and similar conditions, should be investigated to confirm the integrity of the concrete wall and the exposed reinforcing steel areas should be repaired.

- The parapet edge generally requires repair and a damaged fence post support location was noticed in one area along the north wall. See Photo 1.3-23. Roofing work was undertaken at the plant in 2008 under Bid Opportunity No. 428-2008. The parapet was required as part of the bid opportunity and the fence post was fixed.

8.3.11 Oxygen Reactors Nos. 3 and 4

- The exterior rooftop paving slab, in general, exhibits thermal cracking in numerous areas. The slab is nonstructural but will require cracks to be sealed in order to prevent further deterioration.

- Hairline concrete cracks have developed along the reactor tank walls as seen from Gallery No. 3 and Gallery No. 5 south. The crack patterns have both a vertical and horizontal orientation. The horizontal cracks suggest cracking due to bending stresses. The vertical cracks may be attributed to concrete shrinkage. Signs of moisture migration could not be detected. These cracks should be monitored for signs of leakage as part of an on-going maintenance program. Some growth in these cracks exists. See Photo 1.3-24.

- Efflorescence was noted at the lower tank wall of Reactor No. 4 in Gallery No. 5 south. This may suggest moisture migration through the wall. Internal wall coatings and the integrity of the concrete and reinforcing steel should be investigated.

8.3.12 PSA Building

- Horizontal cracks were noted in the exterior wall of the Secondary Blower Room located in the basement. These cracks should be monitored for further propagation.

- Past moisture leakage can be noted on the acoustic ceiling panels over the electrical panel boxes. The roof structure should be investigated for possible sources of leakage. See Photo 1.3-25.

8.3.13 Sludge Thickening and Sludge Truck Bay

- In general, the sludge thickening and sludge truck bay structures appear to be in sound structural condition with no obvious signs of structural distress.

- Limestone veneer requires re-pointing along west side. See Photo 1.3-26. The veneer has undergone some cleaning but was not re-pointed. This work remains outstanding.

- Re-caulking of the parapet cap flashing corner is required.

- 'Bubbling' in the roof membrane at the south end should be investigated and repaired as required. Roof repairs are required as well as flashing replacement. With the damaged flashing and roof membrane, leakage is likely.

- There is damage to the membrane at the pipe supports, which should be investigated and repaired.

8.3.14 Secondary Clarifiers Nos. 1 and 2

- The southwest section of the ground level concrete slab around Secondary Clarifier No. 1 is cracked and is delaminating. See Photo 1.3-27. A hairline cracking pattern coinciding with the reinforcing steel grid is visible. Adequate concrete cover to reinforcing steel is questionable. Cracks also appear at the underside of the slab. The structural integrity of this slab and all slabs around the clarifiers should be investigated and repaired as required. A program to establish extent of delamination in combination with concrete and reinforcement sampling will be required. See Photos 1.3-28, 1.3-30, 1.3-31 and 1.3-33. A significant decrease in the structural integrity could result in high cost repairs in future.

- Steel ties between precast roof sections are not coated. Steel will be susceptible to corrosion. Cleaning, proper preparation, and application of an epoxy coating will protect the steel from corrosion.

- Exposed reinforcing steel in the precast concrete double tee roof beams was noted in the southwest corner of the building (Clarifier No. 2). Column stirrups are also exposed in this area. We recommend that a further investigation to confirm the structural integrity of the roof sections and column be performed and that remedial repair be undertaken as required. See Photos 1.3-29, 1.3-32, 1.3-34, 1.3-35, and 1.3-36. A significant decrease in the structural integrity could result in high cost repairs in future.

- The top two (2) courses of limestone below the cap flashing on the west wall and east wall have mortar deterioration and some loose stones. The stone should have the mortar replaced in these areas, and the cause of this deterioration investigated. See Photo 1.3-38.

- Damaged metal panels on the south end, which should be repaired.

- There is some 'bubbling' of the roof membrane at the southwest parapet and in the central roof area. The cause should be investigated and repaired as required.

- The corners of the parapet flashing on the fan house roof should be re-caulked – Clarifier No. 2.

- 'Bubbling' of the roof membrane on Clarifier No. 1 should be investigated and repaired as required.

- Cap flashing should be re-caulked at the corners.

8.3.15 Secondary Clarifier No. 3

- Maintenance staff indicated that external leakage has been occurring at the northwest corner of the basement wall of Gallery No. 3. It is believed that the leakage is occurring at a pipe penetration through the concrete wall. It is our understanding that the plant staff repaired the leak in 2008.
- A section of hand railing at the north exit is damaged beyond repair and requires replacement. See Photo 1.3-37. The handrail has been repaired since this was noted in 2008.
- The concrete block wall at the southwest stairwell (ground floor level) has cracked. This condition should be monitored for signs of continued cracking.
- There is a loose grating on the north side catwalk, which requires repair.

8.3.16 UV Building

- Some roof ballast is missing from the roof membrane and should be replaced.
- It has been reported by Operations personnel that high interior humidity conditions in the winter have caused “freeze-up” at the entrance door. This condition requires further investigation. Continued moisture migration through the exterior walls may affect the building envelope.

8.3.17 Flushing Water System

During the site review, corrosion of various flushing water piping and equipment was noted. The flushing water lines are in particularly poor condition. The condition of this piping and equipment was discussed with the Plant Supervisor in October 2011. He confirmed that the flushing water piping system was in poor condition and consideration should be given to having it replaced as part of the plant upgrade.

8.3.18 Potable Water System Backflow Prevention

In April, 2009, the City of Winnipeg’s Water and Waste Department Environmental Standards Division undertook an inspection of backflow prevention at the plant. There are two potable water service lines providing water to the SEWPCC. The backflow prevention used on the water service lines was found to be inadequate. It is a zone type of backflow prevention device intended for use on non-potable branches off a potable water system. Thus the backflow prevention devices on the water service lines require upgrading.

8.4 2011 FACILITY CONDITION ISSUES

8.4.1 General

In terms of facility condition assessment, this report is intended to summarize the finding of the past assessment, remedial work completed, and information gaps. There was no intention to undertake a new assessment of the facility. However, through discussions with the supervisor of the SEWPCC, Ron Hahlweg, two new items were identified that need to be addressed as part of any future facility upgrade. These include issues with the flushing water system and backflow prevention system for the facility’s potable water system.

8.5 INFORMATION GAPS

8.5.1 General

Concrete and concrete coatings of the secondary clarifiers, HPO reactors, sludge holding tanks, chambers, channels, etc. could not be assessed due to the tanks being in operation. To assess the integrity of these structures, the concrete and coatings should be evaluated when the contents have been removed and/or unit operation is temporarily shut down. Standard coatings have a life expectancy of 10 to 15 years so depending on the quality of coating used, a cleaning, sandblasting and recoating is likely required.

The exterior limestone veneer, which is the predominant exterior finish for most of the facility, has significant discoloration and apparent mould in areas. This suggests moisture migration through the wall system. The type, cause and remediation should be further investigated. The integrity of the membrane and veneer ties should also be evaluated.

In general, the roofing systems of the structures are in various stages of their life spans. It is recommended that a complete audit of the roofing systems be undertaken.

8.5.2 Building Issues

The Administration Building requires additional investigation to determine the source of the wall leak on the ramp to the grit building. There is also mould reported in the laboratory that requires further investigation.

The Pump and Screening Building dry pit has wet areas requiring investigation and repair. Work is required to determine remedial action required for exposed reinforcing.

The Grit Building concrete slabs need to be tested to determine if moisture migration has affected the structural integrity.

The Maintenance / Service Building appear to have a wall that is shifting. This is displayed through a door issue and cracked veneer. This requires further investigation.

The foundation of the Odour Control Stack could not be checked. A shutdown of the ventilation system should be provided temporarily to permit inspection.

For the Secondary Clarifiers, the structural integrity of this slab and all slabs around the clarifiers should be investigated and repaired as required. A program to establish extent of delamination in combination with concrete and reinforcement sampling will be required. We recommend that a further investigation to confirm the structural integrity of the roof sections and column be performed and that remedial repair be undertaken as required

8.5.3 HPO Reactor Assessment

Assessment of this tank infrastructure asset is important to the preliminary design and associated re-use potential of the tank and would be considered a very high priority. Thus it is very important to "begin planning for the HPO Reactor Inspection immediately." This will prevent delays during the design phase of the upgrade project.

The High Purity Oxygen (HPO) reactors have never been taken out of service for cleaning and inspection. In discussion with Plant Supervisor, Ron Hahlweg, in October 2011, he indicated that HPO Tank No. 1 had once been taken out of service to replace the Reactor 1 Stage 3 R215-MXR mixer. Otherwise, the tanks have not been out of service.

During this singular service outage from August 21, 2004 to September 21, 2004 and Mr. Hahlweg indicated that the treatment quality suffered. The specific treatment quality data was not available [the tank had been out of service]. However this indicates it is important to take an HPO tank out of service at the low flow period (i.e., winter low flows). Additionally only one tank can be taken out of service at a time. The City should consider process adjustments that could improve the treatment capability of the remaining in-service HPO reactors during the maintenance period (i.e., CEPT).

8.5.3.1 HPO Reactor Assessment Preparation

In terms of logistics, Mr. Hahlweg indicated that facility staff would isolate and drain the tank. A third party would be hired for the cleaning process. One company the City contracts for this type of service is Clean Harbours Canada Inc. of Winnipeg.

Prior to any service outage, the City would be required to develop:

- Lock-Out / Tag-Out Procedures
- Safe Work Procedures
- Safe Operating Procedures.

It is proposed that HPO Reactor 3A be investigated. This is one of the tanks that would be re-used, it has good access, and it should be representative of the tank condition for all tanks. Additional consideration should also be given to how best to isolate the discharge of HPO Tank No. 3A form 3B.

8.5.3.2 HPO Reactor Assessment Methodology

The structural investigation scope of work and methodology includes:

- Conduct concrete condition survey of tank wall, roof, and columns by visual and hammer testing
- Evaluate the condition of existing roof membrane system
- Undertake concrete core sampling as required based on the visual inspection
- Evaluate methods for crack repairs and concrete surface restoration including epoxy injection, repair mortars, shotcrete and waterproofing systems, etc.
- Evaluate methods of corrosion protection including epoxy lining, flexible cementitious lining system and rebar protection anodes, etc.

The condition survey would need to be undertaken by Senior Structural Engineers. It is proposed to evaluate a single tank and draw conclusions on the other tanks based on the findings. The tank to be surveyed would need to be cleaned and pressure washed by the City prior to inspection. Temporary fixed and movable scaffoldings within the tank is required to have accessibility up to the underside of tank roof. A visual inspection is proposed.

9.0 Risk Analysis and Mitigation Strategies

9.1 INTRODUCTION

This section summarizes the development and results of a risk assessment conducted on the delivery of the Schematic/Preliminary Design for the South End Water Pollution Control Centre (SEWPCC) expansion and upgrade. The primary objective of this assessment was to identify specific risk related items and possible mitigation measures to minimize or eliminate impacts that could jeopardize the successful delivery of the Preliminary Design phase of this project. The risk items identified as part of this activity have been categorized to help rank and prioritize actions to support the cost-effective and timely delivery of the technical aspects associated with the Preliminary Design.

The risk assessment includes the identification and scoring/ranking of vulnerabilities, and the development of mitigation strategies related to key design and delivery factors associated with the scope of work for the Schematic Preliminary Design. The identified risks encompass technical aspects as well as related project management factors, some of which are contained in other technical memoranda and noted as risks or assumptions as part of the SEWPCC Project Definition/Validation assignment. Ownership of the risk and associated mitigation strategy is premised on the basis of the party best capable of managing and controlling the risk.

9.2 RISK ASSESSMENT FRAMEWORK

A key component of risk management is the identification of possible risk events and the quantification of their likelihood of occurring (“the probability”) and their impact if the event took place (“the severity”). The product of probability and severity is the assessed risk of an event taking place. The following definitions of terms have been used to place the assessment into context.

- Uncertainty: Multiple outcomes, insufficient data to predict a future outcome with any degree of accuracy.
- Event: What could happen?
- Probability: How likely is the event to happen < 100 percent?
- Consequence: What will take place if the event happens?
- Severity: The impact of the consequence.
- Risk: An adverse event taking place.
- Mitigation: Measures to reduce the Probability or Impact.
- Exposure = Risk - Mitigation

It is important to note that a risk becomes an issue if the probability has a likelihood of 100 percent chance of occurring. Specifically, it is no longer a risk but rather an event that will occur and measures need to be implemented to control its impact. Otherwise, the satisfactory completion of the component it influences may jeopardize the overall successful delivery of the project, if not dealt with in an effective and timely manner.

The Program Team have developed a risk and opportunity (R&O) framework for the overall delivery of the SEWPCC upgrade and expansion project. The assessment done for the successful delivery of the Preliminary Design uses the elements contained in the R&O framework prepared by the Program Team so that this risk assessment can be directly assimilated into the overall risk register for this project.

9.3 PROJECT DRIVERS

To provide a consistent content for the risk assessment, the key project drivers need to be known. For example, if schedule is a key driver, then a delay in schedule could have a severe impact on the project and would be ranked with a high severity. A list of typical project drivers was supplied to the Program Team at the workshop held on December 5, 2011 for review and ranking. The list was reduced and refined to be specific to this aspect of the project. As a group, the refined list was ranked to provide an overall context to rate the severity of an event. The following list, in order of priority, was developed as part of the workshop. It should be noted that this is a relative ranking for the drivers noted below:

1. **Cost Certainty**
2. **Lowest Whole Life Cost**
3. **Schedule**
4. Design/Performance Confidence
5. Owner Control
6. Stakeholder Impact
7. Risk Transfer

It was acknowledged that the final ranking of project drivers required input from addition stakeholders. To that end, it was recommended that the Program Team deliberate on the list and ranking to firmly establish the key drivers so that the risks can be confidently prioritized and dealt with accordingly in the Preliminary Design. The severity of the risks were based on the above noted driver ranking and scored according.

9.4 RISK ASSESSMENT

A scoring framework was prepared by the Program Team for use in the quantification of the likelihood/probability of risk occurrence, the magnitude of the risk, and the scoring of severity (refer to Tables 9.1, 9.2, and 9.3). A list of possible risk items associated with the successful delivery of the SEWPCC upgrade and expansion were prepared by Stantec. The list of items were reviewed at the workshop held on December 5, 2011. Each risk item was reviewed to confirm is applicability and relevance to the project. The risk items were clarified where necessary, combined where appropriate, or removed if the item was deem as either 100% likely to occur (i.e., no longer a risk but an item for design consideration), an uncertainty (i.e., risk could not be quantified), or outside of the scope of the current assignment (e.g., a design issue or detail to be dealt with in subsequent engineering design phases). The workshop attendees (see Appendix C) were then placed into smaller working groups to evaluate and score the risks. Each group provided their scoring and supporting reasons if there was a major difference between the scores from other groups. The overall group deliberated on the scoring, and by consensus, arrived at a final ranking and rating of each risk item, along with its owner (i.e., the party best capable of managing the risk).

A draft table of risks, their initial scoring, and possible mitigation measures were provided to the Program Team in advance of the workshop held on December 5, 2011. The workshop was structured to permit a review of the risks, revisions to the scoring as required and clarification/direction on possible mitigation measures to reduce or eliminate the risk. Some of the risks were determined to be design issues and were accordingly transferred to the scope of work associated with Preliminary Design. A revised risk assessment table was provided to the Project Team after the workshop for their review and comment before finalizing the assessment.

9.5 SUMMARY

Table 9.4 summarizes the final outcome of the risk assessment, including the party best capable of managing and mitigating the risk.

An important aspect associated with the Risk Assessment is the timely mitigation of identified risks. As such, certain items will need to be resolved ahead of others because of their influence on schedule, and the development of scope of work associated with the Preliminary Design. This is intended to be a living document which is to be updated at regular intervals as the project proceeds.

Table 9.1 - Assessing Liklihood/probability of Risk Occurrence

Descriptor	Rating	Frequency	Probability
Almost certain	5	Is expected to occur during projects of this type	> 95%
Likely	4	More likely as not, regularly occurs during projects of this type	60% < x < 95%
Moderate	3	As likely as not, might occur at sometime during a project of this type	30% < x < 60%
Unlikely	2	Could occur at some time during the project, rarely occurs on projects of this type	5% < x < 30%
Rare	1	Only occur in exceptional circumstances on projects of this type	< 5%

Note on the use of Specific Probability Data and Distributions:

The first step in assessing the likelihood / probability of a risk should always be to apply the project teams engineering judgement and experience, in most cases this approach is all that is required. Specific probability data is available from a variety of sources, however unless the assumptions underpinning such distributions and data hold, the results can be misleading and introduce greater risk. Such data should be checked carefully before it is used.

Source: Veolia and City of Winnipeg, Dec 2011

Table 9.2 - Assessment Framework to Quantify Risk and Opportunity

Assessment of the Magnitude of Opportunity

	Insignificant Savings 1	Minor Savings 2	Moderate Savings 3	Major Savings 4	Significant Savings 5
Cost	Cost savings <$10K	Cost Savings <$100K	Cost Savings <$1M	Cost savings <$10M	Cost savings >$10M
Time	Time savings <½ day	Time savings ½ – 1 day	Time savings >1 day, < 1 week	Time savings >1 week, < 1 month	Time savings >1 month
Other??					

Assessment of the Magnitude of Risk

Descriptor	Negligible	Moderate	Substantial	Severe	Disastrous
	Small effect on costs	Moderately effects costs	Considerably affects cost	Serious threat to the organization, public etc.	The impact is totally unacceptable to the
	1	2	3	4	5
Safety *	Negligible – No injury, near miss	Minor – minor cuts, bruises, muscle strain	Serious – broken bones, muscle and ligament injuries	Serious / permanent injury / illness	Catastrophic – Single or Multiple fatalities
Financial Impact upto a maximun value (re-work / loss etc..)	≤ $10,000	≤ $100,000	≤ $1,000,000	≤ $10,000,000	>$10,000,000
Financial Impact % of Target Cost	Do not use at the moment				
Schedule, impact on critical path*	Not likely to impact dates	Likely to absorb float between planned dates and target dates	≤ 1 month	≤ 2 month	> 2 month
Environment *	Negligible Environmental effect	Nuisance / minor but reversible Environmental harm	Moderate but short term Environmental harm	Localised, long term Environmental harm	Extensive long term Environmental harm
Regulatory *	negligable, near miss	report required to regulatory body	Inspection by Manitoba Env safety officer etc..	CEC review	Clean Environment Commission (CEC) Hearing
Image / Reputation *	Single Public Enquiry	Multiple Public Enquiries and / or informal Councillor and / or MP Request	Moderate Media Political – Formal Council and / or MP Request / Moderate Public Impact	Provincial Government, Major Political & Media Scrutiny / Major Public Impact	Federal Investigation
Moral	No Impact	Grumblings at wter cooler	Moderate / Increasing Absenteeism	Major Negative / Loss of Staff / "Go Slow"	Catastrophic Negative / walk out
Legal	No Liability	Written Claim Damages < $10,000	Damages > $10,000 < $250,000	Damages >$250,000 < $1,000,000	Damages >$1,000,000
Other *					

Source: Veolia and City of Winnipeg, Dec 2011

Table 9.3 - Framework to Assess Risk Severity

Total Severity	***Category***	***Response***
20-25	Critical	Expected cost to the project is unacceptably high. This risk must be eliminated or transferred before proceeding with the project. Attempt to avoid or transfer risk
10-20	Serious	Expected cost is high compared to total project cost. It probably is cost effective to eliminate or transfer this risk.
5-10	Important	Consider eliminating or transferring. If accept then manage proactively.
0-5	Acceptable	Accept and manage

Source: Veolia and City of Winnipeg, Dec 2011

TABLE 9.4

Risk Register

Risk Category	Risk Item (Cause Event)	Specific Consequence (this may occur)	The Effect	Driver(s)	Likelihood	Severity	Rating	Mitigation	Owner
	Undefined Project Drivers	Unresolved focus can lead to design mismatches	Extra costs and time	1, 3, 6	5	5	25	Clearly define project drivers at start of Preliminary Design	PT
	Internal resistance to adopt new process controls strategy, and transition plan	Internal resistance to change from a DCS to a PLC system can delay the overall design	Can stall the project an result in schedule delays	3, 5, 6	4	5	20	Resolve internal disputes on a PLC based control system early in the preliminary design process	PM
	Pre-selection of BAF and HRC not undertaken early in the design process	Delaying this critical path item can extend timeline if not dealt with as a priority item early in the delivery process.	Extends schedule completion date	3	5	4	20	Confirm that selection and procurement process for BAF and HRC early in the Preliminary design process	PM
	Evolving Regulatory compliance requirements for effluent quality parameters and averaging periods	Changing License requirements can result in the need to rework design to meet compliance requirements	Lost time, and additional budget to complete the design	1, 3, 6	4	4	16	Confirm License requirements before starting Preliminary Design and the design criteria are aligned with License requirements	PT
	Undefined Project Delivery Method	Influences the scope and direction of the project and inability to get suppliers to support design development	Negatively impact quality and increase budget requirements	1, 5	5	3	15	Confirm delivery method at start of Preliminary Design	PM
	Unachievable Regulatory compliance requirement	Deadline 31-Dec-2012 not achievable which could force set unrealistic schedule to complete the design	Negatively impact quality, resource management, and review timeframes	3, 4, 6	5	3	15	Negotiate new in service date with Regulators	PT
	Unknown condition of mechanical and electrical assets	The functional life of some components may be near or past their safe and reliable operating status	Additional cost to replace or rehabilitate assets	1, 2	3	4	12	PT to confirm the condition of these assets	Designer
	Poor communications affecting clients needs and timely decisions	Delay in making decisions and providing input	Could impact the quality, cost and schedule of the project	3, 4	3	4	12	Develop a communication plan and protocol for effective exchange of information and validation of decisions for timely resolution of key deliverables	PM
	Stantec resources not available when required	Not available as need to meet timelines	Can result in project schedule delays	3	2	5	10	Stantec to provide clear project plan and resource loaded schedule	Designer
	Condition of HPO reactors unknown	Tanks may be in a highly deteriorated state and require extensive repairs in order to be reused for other purposes	Additional cost to rehabilitate tanks	1	3	3	9	Investigate condition of HPO tanks ASAP to determine condition and budget requirements	Designer

TABLE 9.4

Risk Register

Risk Category	Risk Item (Cause Event)	Specific Consequence (this may occur)	The Effect	Driver(s)	Likelihood	Severity	Rating	Mitigation	Owner
	Council may not support elimination of HLW from SEWPCC	Additional design work required	Result in process upset or additional treatment required incur extra design, schedule and budget.	1, 3, 4, 6	2	4	8	Resolution of HLW acceptance at the start of the Preliminary design to confirm the need to build in processes and operations to protect the biological processes.	PT
	Decision to change or remove solid processing at the SEWPCC	Processes and technologies to be used to digest and dewater sludge, and treat the reject water can have a major impact on the design process required to achieve effluent limits. Unresolved end use of solids or disposal and mitigation requirements for odor control.	Potentially additional work to confidently prepare detail designs, scope, budget and schedule implications	1, 3,	2	4	8	Decision on solids process at SEWPCC required ASAP	PT
	Potential for scope creep due to vague work definition or out-of-scope items	As the project progresses, the project needs might evolve	Negatively impact the schedule, budget (and successful delivery of the project).	1, 3, 6	2	4	8	Identify additional items as early as possible, or items that can cause a change in the focus and direction of the preliminary design.	PM
	Loss of continuity of key project team members	The loss of key members and their associated project knowledge	Could impede the progress of the project resulting in schedule delays and additional costs	3	2	4	8	Develop a project succession plan for all key project staff and have a supporting deputy designated to all key project staff	All
	(Uncertainty) Under or over-rate treatment capacity of primaries to converted to CEPT	Unresolved treatment capacity could result in over building or under building additional HRC, unknown increase in solids quantity and quality, handling of solids and digestibility, FOG removal	Least total cost - Potentially expend more money than required for clarification or insufficient clarification for BAF to preform well	2	2	3	6	Stress test of primary clarifier to assess safe and reliable CEPT performance.	Designer
	Insufficient level of details provided to cost consultant required to develop target cost	Additional design work required	schedule, decrease in cost certainty	1, 3	2	3	6	Engagement of cost consultant early in Preliminary Design process	Designer / PM
	Project definition not based on current draft licences	Changing design requirements may result in additional scope of work	schedule and increased design cost	1, 3	1	3	3	Early communication with Regulatory authorities to validate compliance requirements.	PT
	Evolving Regulatory compliance requirements for collection system may drive more WWF to plant	Reduction of SSOs from collection system can drive more WWF to plant to for treatment	Increased risk of non-compliance with effluent limits and incur regulatory penalties	1, 6	1	3	3	Plan for more WWF delivered to plant or WWF treatment in collection system	PT

TABLE 9.4

Risk Register

Risk Category	Risk Item (Cause Event)	Specific Consequence (this may occur)	The Effect	Driver(s)	Likelihood	Severity	Rating	Mitigation	Owner
	Evolving Regulatory compliance requirement to eliminate plant by-passes	By-pass not related to emergency operation may not be allowed and require that all flows receive a minimum level of treatment	Risk of non-compliance with compliance requirement and incur regulatory penalties	1, 6	1	3	3	Provision for primary treatment and disinfection of all flows	PT

SEWPCC Project Definition / Validation
Project# 111213121

Issues Log

3/23/2012

Category	Issue	Specific Consequence (this will occur)	The Effect	Likelihood	Severity	Rating	Mitigation Action	Owner
task	Basis of Design for other major equipment, e.g., Screens, grit removal	Potentially requires redesign of HRC and BAF	negatively impact schedule and increase in budget requirements	3	4	12	Supplier to provide appropriate level of detail for design and development of specifications	Designer
	Inadequate training of operators prior to testing and commissioning of the facility	Operating staff may not be in a position to confidently operate the new systems because training was delayed due to resolution of supplier provided equipment and recommended operating practices.	Operators unprepared for commissioning resulting in need for addition time to and budget.	3	4	12	Pre-select BAF and HRC early in the design process to allow sufficient time to prepare a staffing plan, prepare a training schedule, and conduct a detailed review of the operating strategy	
	Undefined WWF Disinfection method	Resolution of choice of technology required in order to develop appropriate disinfection systems, otherwise its resolution will delay this aspect of the preliminary design	Negative impact schedule	2	5	10	Investigate alternative disinfection options (e.g., expand existing UV) or increase failsafe measures	
Issue	Unknown priority of future allowance for Biological P removal	Reworking design based on process and choice of treatment technology	Scope changes and associated budget and schedule increases	2	3	6	Clear direction from PT required early in the Preliminary Design process	
	PT decides to move forward with P Recovery late in the preliminary design	Reworking design based on process and choice of treatment technology	Scope changes and associated budget and schedule increases	3	2	6	Linked to decisions associated with Bio-P and digestion	
task	Undefined and quantified odor sources	Complaints from surrounding developments may cause Regulatory to order the implementation of an appropriate odor measures to achieve acceptable levels	Increased risk of non-compliance with air quality requirements, extra time and cost to implement control measures, and incur regulatory penalties	3	2	6	Assess as part of preliminary design	

Definitions

Risk Categories
Execution
Integration
White Space

Project Drivers:

Rank	Name
1	Cost Certainty
2	Lowest Total Cost
3	Schedule
4	Design / Performance Certainty
5	Owner Control
6	Stakeholder Influence
7	Risk Transfer

Critical Success Factors (for mitigation of risks)

1. Cost Certainty
Scope definition
Schedule definition
What-if scenario planning
Risks quantified / contingency
Existing asset risk

2. Schedule
Resource capacity
Timely decision making
Scope definition (Regulatory requirements / "good enough" line
Defined critical path / schedule

3. Lowest Total Cost
Scenario definition (capex / opex)
Dynamic cost modelling
Existing asset risk

10.0 References

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11.0 Acronyms and Abbreviations

AA – Annual Average

AS BNR – Activated Sludge Biological Nutrient Removal

BAF – Biologically Active Filter

BOD_5 – Five Day Biochemical Oxygen Demand

BW - Backwash

C – Carbon

CBOC – Carbonaceous Biochemical Oxygen Demand

CCME – Canadian Council of Ministers of the Environment

CEPA – Canadian Environmental Protection Act

CEPT – Chemically Enhanced Primary Treatment

COD – Chemical Oxygen Demand

DC – Direct Current

DN – Denitrification

EDC – Endocrine Disrupting Chemicals

GM – Geometric Mean

HLW – Hauled Liquid Waste

HH – House Hold

HPO – High Purity Oxygen

HRC – High Rate Clarification

HRT – Hydraulic Retention Time

MLSS – mixed Liquor Suspended Solids

MOE- Ministry of Environment

MOP – Manual of Practice

N - Nitrogen

NE – North End

NDN – Nitrification-Denitrification

NEWPCC – North End Water Pollution Control Centre

NH_3 – Ammonia

NO_2/NO_3 – Nitrate-Nitrite

NPSH – Net Positive Suction Head

NPV – Net Present Value

Ortho-P – Soluble Phosphorous

Part-P – Particulate Phosphorous

PSA – Pressure Swing Adsorption

PDR – Project Definition Report

PSR – Process Selection Report

PST – Primary Settling Tank

SBR – Sequencing Batch Reactor

SE – South End

SEWPCC – South End Water Pollution Control Centre

SHT – Sludge Holding Tank

SLR – Solids Loading Rate

SOC – Soluble Organic Carbon

SOR – Surface Overflow Rate

TOC – Total Organic Carbon

TKN – Total Kjeldahl Nitrogen

TSS – Total Suspended Solids

TP – Total Phosphorous

VFA – Volatile Fatty Acids

USEPA – United States Environmental Protection Agency

UV - Ultraviolet

VFD – Variable Frequency Drive

VS – Volatile Solids

VSS – Volatile Suspended Solids

WE – West End

WEF – Water Environment Federation

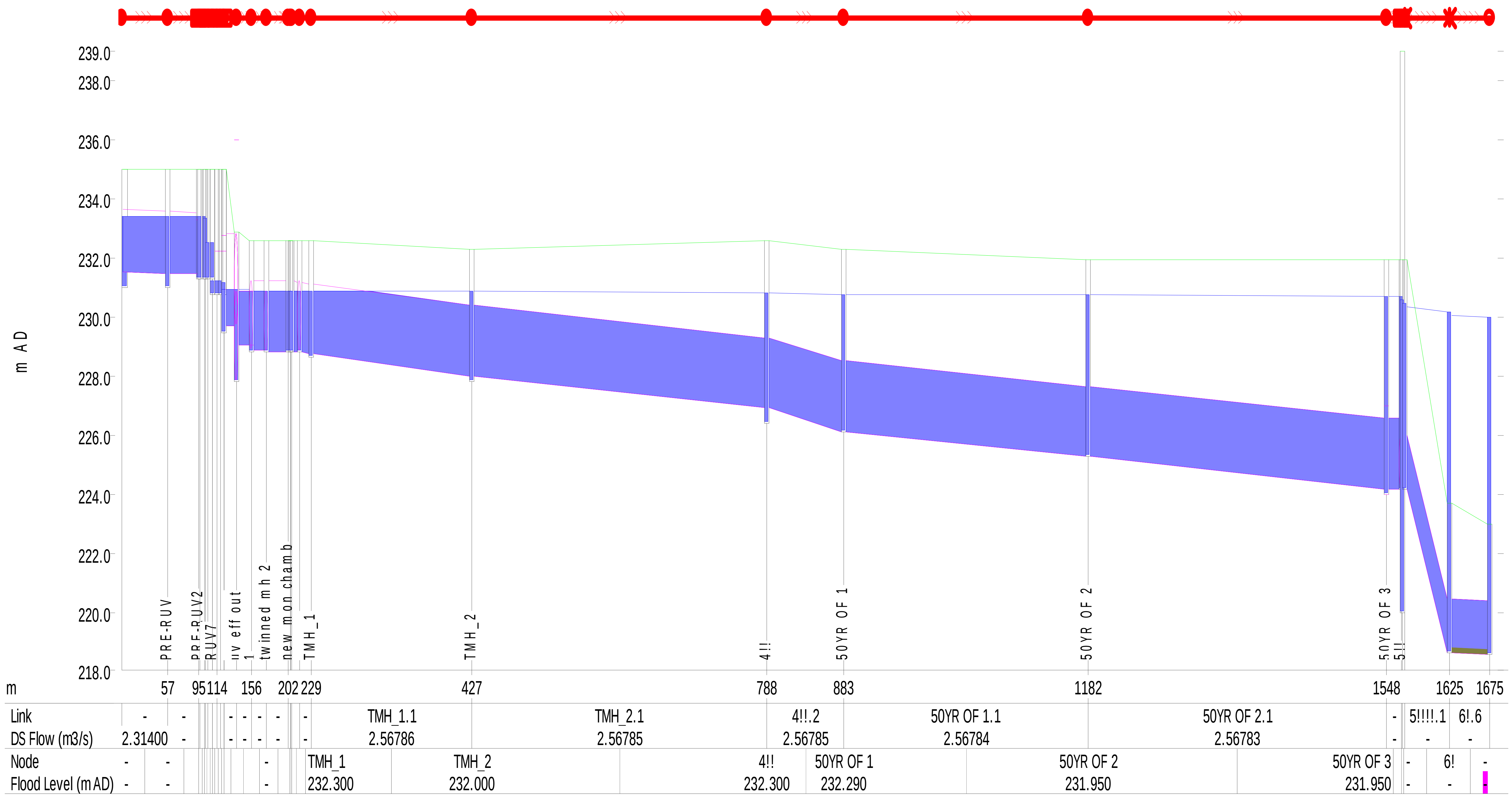
WERF – Water Environment Research Foundation

WEWPCC – West End Water Pollution Control Centre

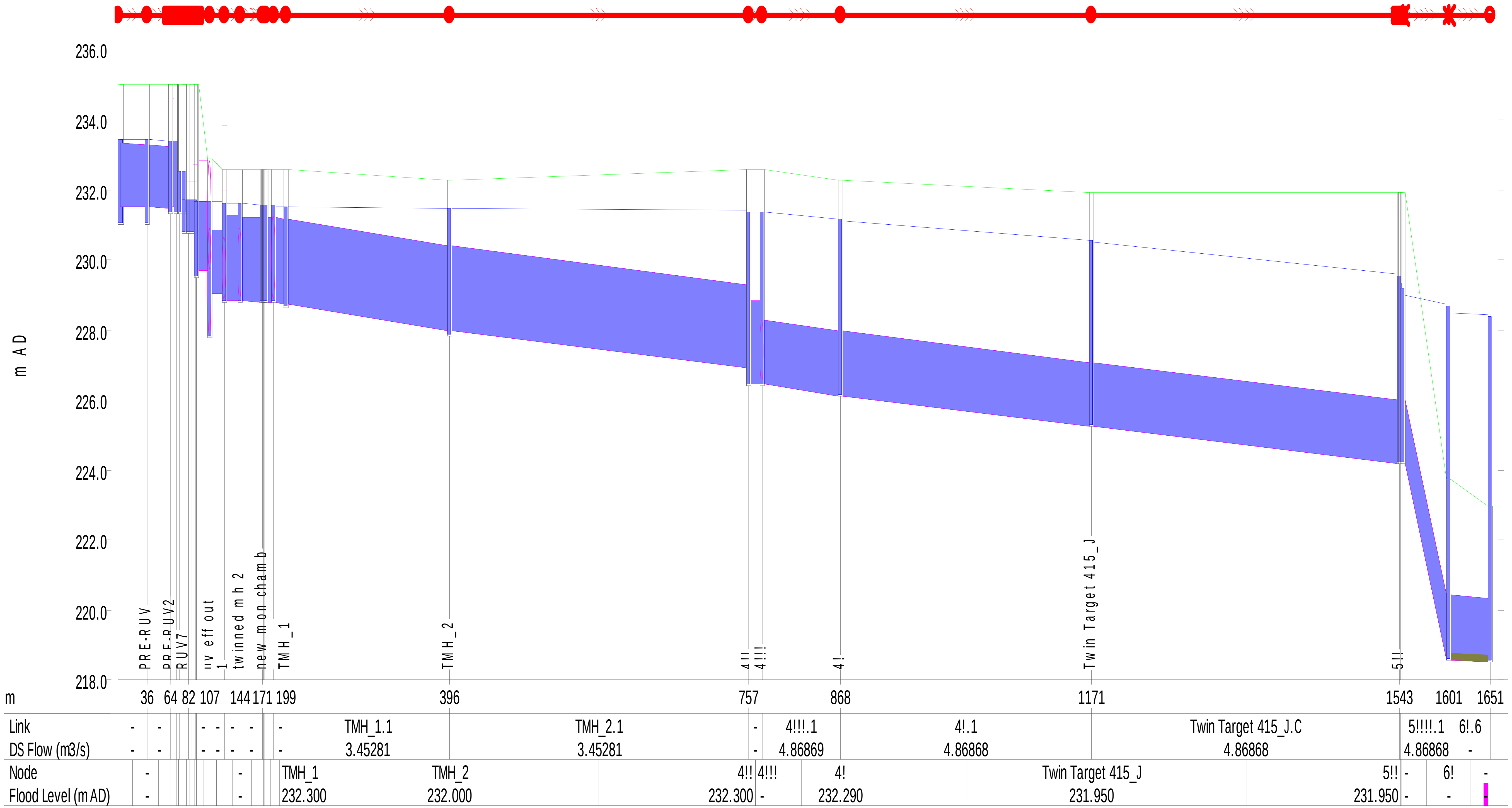
WWTP – Wastewater Treatment Plant

APPENDIX A

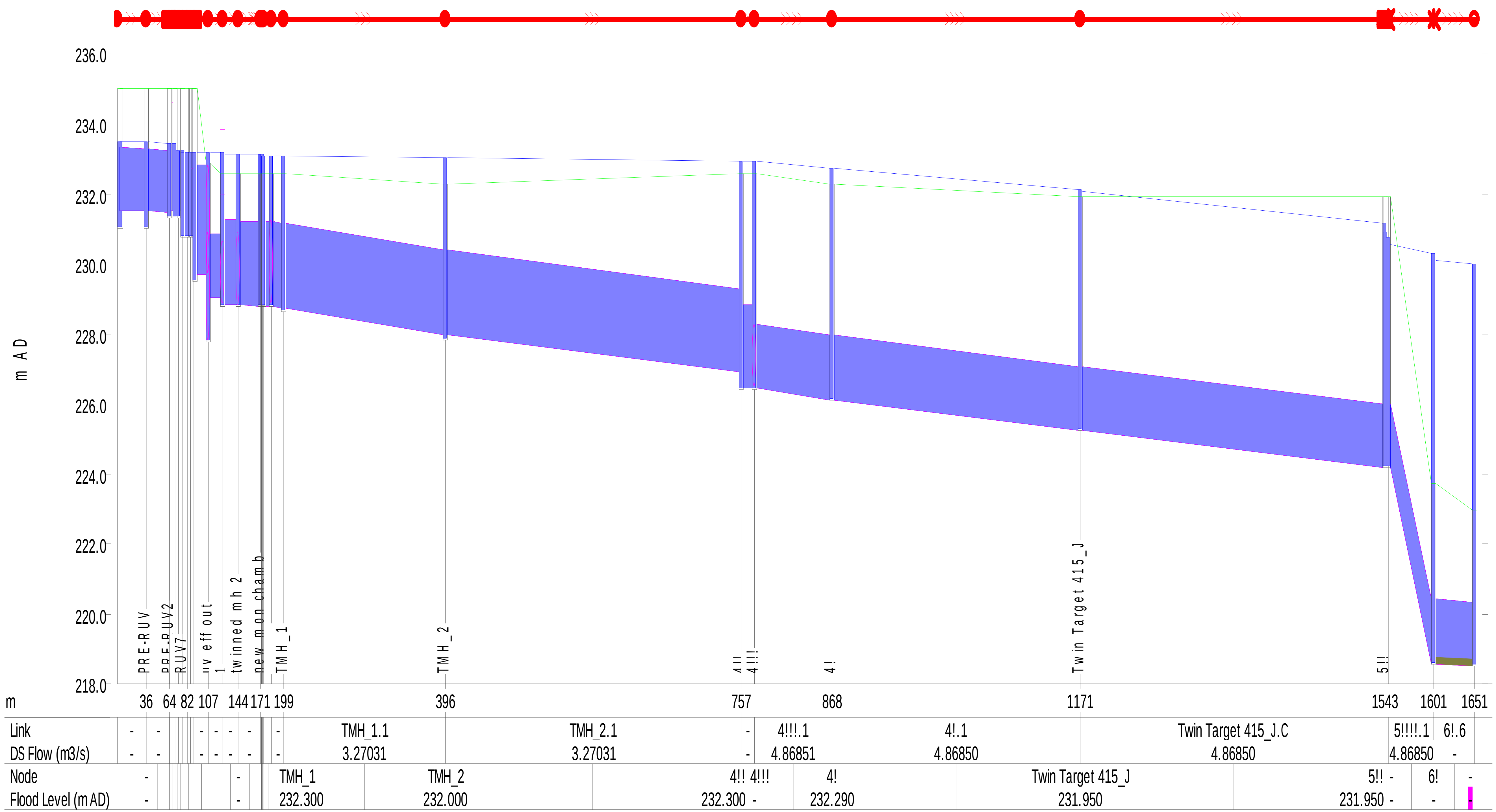
Hydraulic Modeling Output



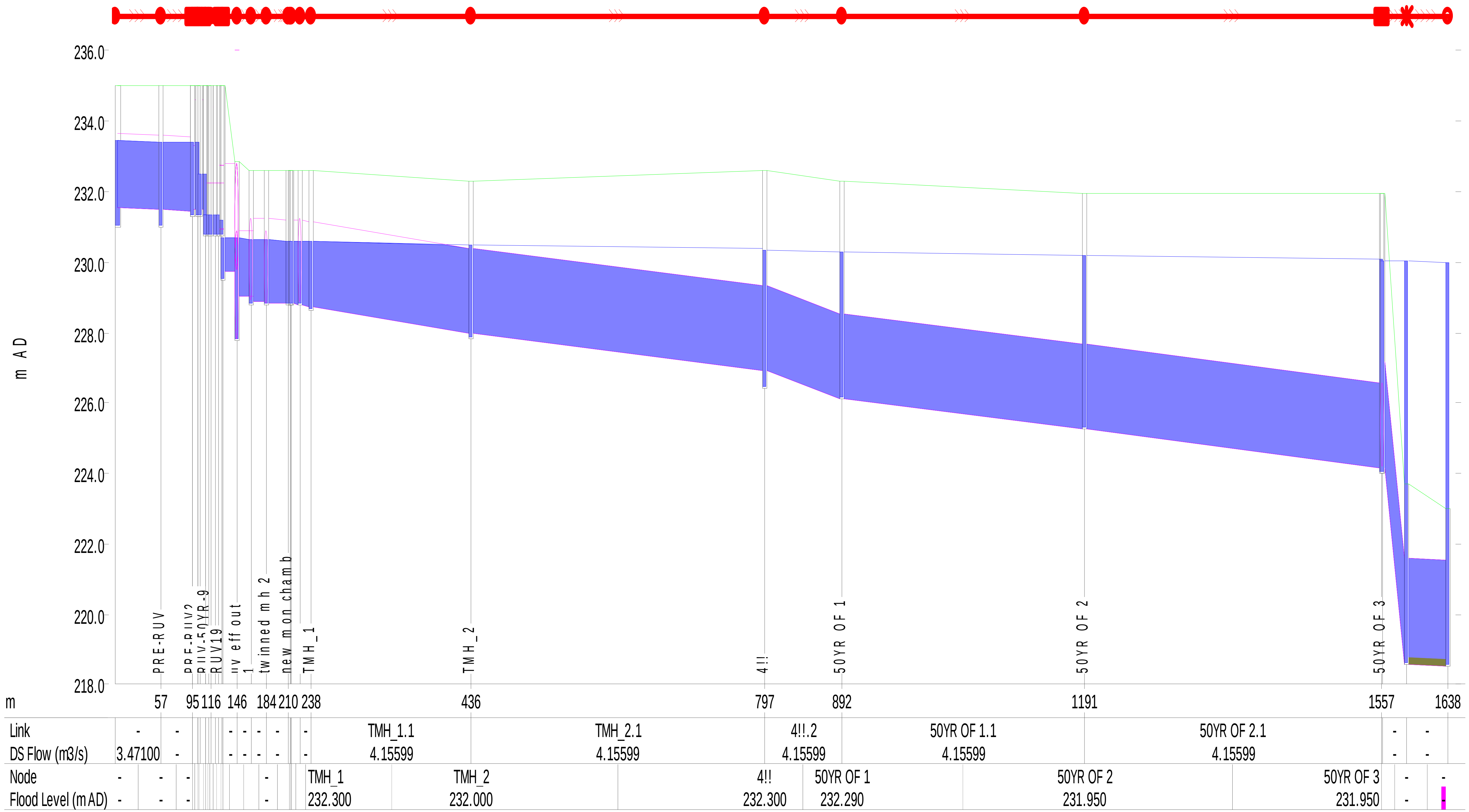
Just Upstream of Raised UV Bldg. to Red River - 325 MLD with River at 230.00m



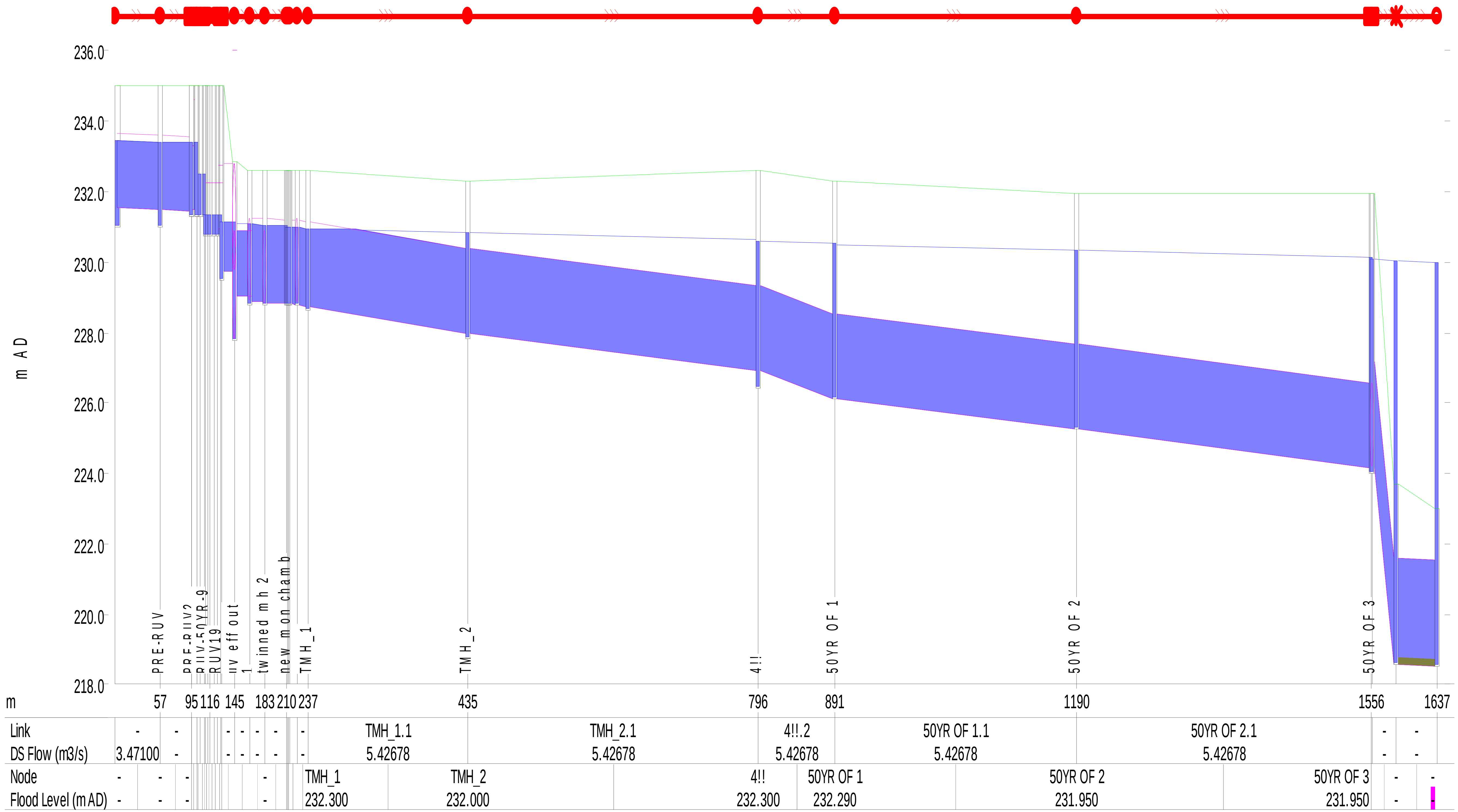
Just Upstream of Raised UV Bldg. to Red River - 420 MLD with River at 228.72m



Just Upstream of Raised UV Bldg. to Red River - 420 MLD with River at 230.00m



Just Upstream of Raised UV Bldg. to Red River (Expanded Outfall) - 525 MLD with River at 230.00m



Just Upstream of Raised UV Bldg. to Red River (Expanded Outfall) - 680 MLD with River at 230.00m

APPENDIX B

Condition Assessment Photographs

All photos were taken in the spring of 2008 as part of the Civil Condition Assessment Work.

Photo 1.3-1 – GENERAL
Limestone Veneer Staining – Apparent Mould noted in 2008
In June 2009, cleaning work was undertaken under Work Order 0904280. Additional cleaning is required and Work Order 1004092 was issued in May 2010. The cleaning work is yet to be completed.

Photo 1.3-2 - ADMINISTRATION BUILDING AND EXPANSION
Hairline Crack – North Entrance Entry Slab noted in 2008
The entry slab was patched in June 2009.

Photo 1.3-3 – ADMINISTRATION BUILDING EXPANSION
Apparent Mould on Wall in Laboratory Storage Room noted in 2008.
The mould has been cleaned and the walls painted. There is no indication that the cause of the mould has been addressed.

Photo 1.3-4 – ADMINISTRATION BUILDING AND EXPANSION
Deterioration of Caulking at Parapet Flashing
This roof leak has since been repaired under Work Order 1106899

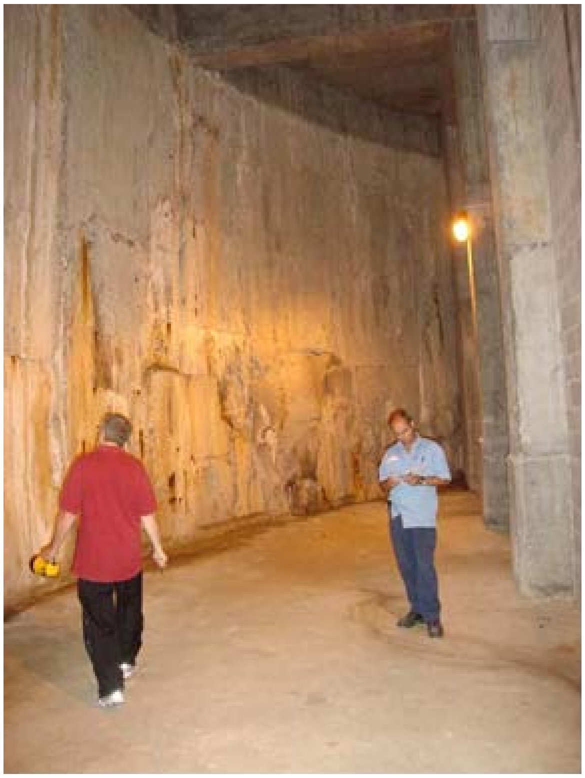


Figure 1.3-5 - PUMP AND SCREEN BUILDING
Efflorescence on Ring Wall – Dry Well
This issue has not been addressed since being noted in 2008.

Photo 1.3-6 – PUMP AND SCREEN BUILDING
Efflorescence on Ring Wall – Opening in Stairwell
This issue has not been addressed since being noted in 2008.

Photo 1.3-7 – PUMP AND SCREEN BUILDING
Exposed Reinforcing Steel at base of Ring Wall – Dry Well
This issue has not been addressed since being noted in 2008.

Photo 1.3-8 – PUMP AND SCREEN BUILDING
Exposed Reinforcing Steel at Base of Ring Wall – Dry Well
This issue has not been addressed since being noted in 2008.

Photo 1.3-9 – PUMP AND SCREEN BUILDING
Exposed Reinforcing Steel – Top of Dry Well
This issue has not been addressed since being noted in 2008.

Photo 1.3-10 – PUMP AND SCREEN BUILDING
Exposed Reinforcing Steel at Entry Slab
This issue has not been addressed since being noted in 2008.

Photo 1.3-11 – GRIT BUILDING
Efflorescence on Walkway Slab
This issue has not been addressed since being noted in 2008.

Photo 1.3-12 – GRIT BUILDING
Spalled Concrete at Guardrail in Screen Room
This issue has not been addressed since being noted in 2008.

Photo 1.2-13 – GRIT BUILDING
Spalled Concrete at Drain
This issue has not been addressed since being noted in 2008.

Photo 1.3-14 – GRIT BUILDING
Exposed Reinforcing Steel at Concrete Stair in Grit Bin Area
This issue has not been addressed since being noted in 2008.

Photo 1.3-15 – GRIT BUILDING
Damaged Parapet Flashing
The roofing and flashing was replaced in 2008 under Bid Opportunity No. 428-2008.

Photo 1.3-16 – GRIT BUILDING
"Bubbling" of Roof Patches
The roofing and flashing was replaced in 2008 under Bid Opportunity No. 428-2008.

Photo 1.3-17 – GRIT BUILDING
Damaged Skylight
The Skylight was replaced as part of the roof replacement in 2008 under Bid Opportunity No. 428-2008.

Photo 1.3-18 – MAINTENANCE/BOILER BUILDING
Floor Crack Pattern – Workshop Area
This issue has not been addressed since being noted in 2008.

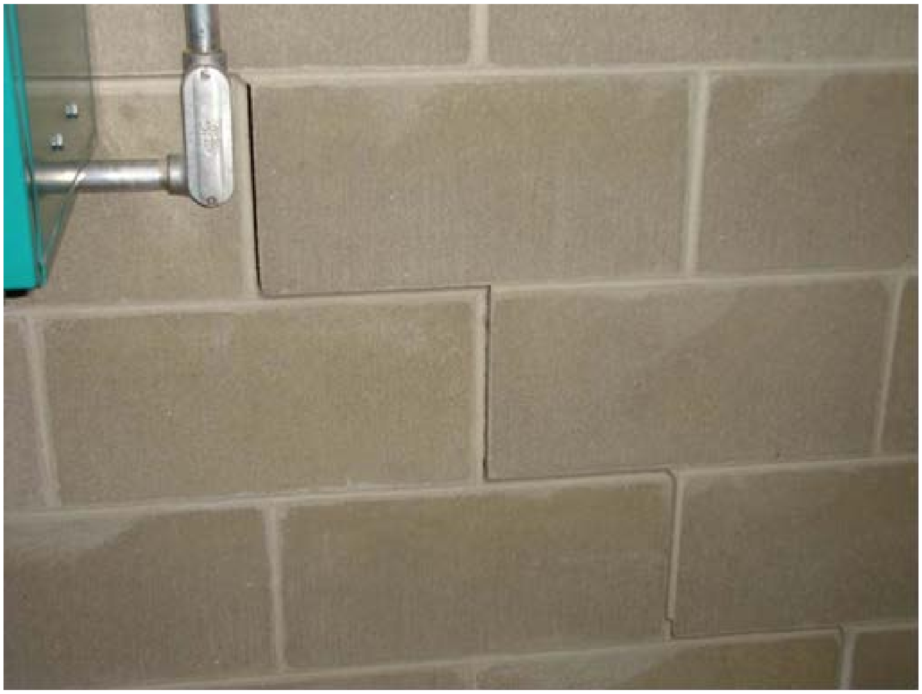


Photo 1.3-19 – MAINTENANCE/BOILER BUILDING
"Ladder" cracks in Concrete Block Partition – Storage Room
This issue has not been addressed since being noted in 2008.

Photo 1.3-20 – PRIMARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Link to Grit Building – North Elevation – East End
In June 2009, cleaning work was undertaken under Work Order 09042880. Additional cleaning is required and Work Order 1004092 was issued in May 2010. The cleaning work is yet to be completed.

Photo 1.3-21 – PRIMARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Exposed Reinforcing Steel at Base of Concrete Tank
This issue was addressed under Work Order 1005511, issued in 2010.

Photo 1.3-22 – PRIMARY CLARIFIER NO. 3 STRUCTURE
Damaged Concrete – Entry Slab South East Corner
This issue has not been addressed since being noted in 2008.

Photo 1.3-23 – OXYGEN REACTORS NOS. 1 AND 2 STRUCTURE
Damaged Parapet – North Face
The roofing and flashing was replaced under Bid Opportunity No. 428-2008. The parapet was repaired as part of this work.

Photo 1.3-24 – OXYGEN REACTORS NOS. 3 AND 4 STRUCTURE
Typical Vegetation Growth – Roof Top
This issue has not been addressed since being noted in 2008.

Photo 1.3-25 – PSA BUILDING
Water Stained Acoustic Ceiling
This issue has not been addressed since being noted in 2008.

Photo 1.3-26 – SLUDGE THICKENING AND SLUDGE TRUCK BAY STRUCTURE
Southwest Corner of Sludge Thickening
In June 2009, cleaning work was undertaken. Additional cleaning is required and Work Order 1004092 was issued in May 2010. The cleaning work is incomplete.

Photo 1.3-27 – SECONDARY CLARIFIER NO. 1 STRUCTURE
Crack Pattern – Southwest Corner Clarifier No. 1 Walkway
This issue has not been addressed since being noted in 2008.

Photo 1.3-28 – SECONDARY CLARIFIER NO. 1 STRUCTURE
Delaminated Concrete – Southwest Corner Clarifier No. 1 Walkway
This issue has not been addressed since being noted in 2008.

Photo 1.3-29 – SECONDARY CLARIFIER NO. 2 STRUCTURE
Exposed Concrete Column Tiles – South End Clarifier No. 2
This issue has not been addressed since being noted in 2008.

Photo 1.3_30 – SECONDARY CLARIFIER NO. 2 STRUCTURE
Fan Room – South End
This issue has not been addressed since being noted in 2008.

Photo 1.3-31 – SECONDARY CLARIFIER NO. 2 STRUCTURE
Exposed Reinforcing Steel – Precast Roof Section – South End
This issue has not been addressed since being noted in 2008.

Photo 1.3-32 – SECONDARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Stained Concrete Column – Gallery No. 3
This issue has not been addressed since being noted in 2008.

Photo 1.3-33 – SECONDARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Exposed Reinforcing Steel – Base of Concrete Tank Wall
This issue has not been addressed since being noted in 2008.

Photo 1.3-34 – SECONDARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Crack Along Underside of Beam – Gallery No. 3
This issue has not been addressed since being noted in 2008.

Photo 1.3- 35 – SECONDARY CLARIFIERS NOS. 1 AND 2 STRUCTURE
Cracks in Concrete Block – Stairwell
This issue has not been addressed since being noted in 2008.

Photo 1.3-36 – SECONDARY CLARIFIER NOS. 1 AND 2 STRUCTURE
Corrosion at Roof Drain
This issue has not been addressed since being noted in 2008.

Photo 1.3-37 – SECONDARY CLARIFIERS NO. 1 STRUCTURE
Broken Handrail – North Entrance noted in 2008.
The handrail has been replaced.

Photo 1.3-38 – SECONDARY CLARIFIERS NOS. 1 AND 2 STRUCTURE
Loose Limestone Veneer in Upper Courses at Roof Parapet
This issue has not been addressed since being noted in 2008.

APPENDIX C

Project Risk and Opportunity Register

APPENDIX C – Workshop Attendees

Veolia

- Bruno Valla
- Americ Simon

City of Winnipeg

- Jackie Veilleux
- Neil Harrington
- Dwight Gibson
- Ron Hahlweg
- Rene Grosselle
- Ron Amann

Stantec

- Carmine Militano (facilitator)
- Carlos Vieira
- Jamie Brewster
- Nick Szoke
- Saibal Basu
- Eric Wiens

Agenda

Stantec

SEWPCC Project Definition/Validation
Risk Analysis and Mitigation Strategies Workshop

Training Rooms A & B – 1199 Pacific Avenue
December 5, 2011

Time:	Item:
9:00-9:10am	1) Introductions
9:10-9:15am	2) Purpose of Workshop
9:15-9:20am	3) Definition of Terms
9:20-9:35am	4) Workshop Framework
9:35-11:00am	5) Risk Identification & Evaluation
≈10:30-10:45am	15 minute Coffee Break
11:00-11:25am	6) Ranking of Risks – Highest to Lowest
11:25-11:45am	7) Risks to be addressed in Preliminary design
11:45am-12:00pm	8) Approach to Finalizing Risk Assessment

APPENDIX D

Excerpts from SEWPCC Preliminary Design Report

SEWPCC Upgrading/Expansion
Preliminary Design Report

SECTION 11 - WET WEATHER TREATMENT

Table of Contents

11.0 WET WEATHER TREATMENT....................................11.1
11.1 INTRODUCTION....................................11.1
11.2 BASIS OF ANALYSIS....................................11.3
11.2.1 Maximum Day Flows for Wet Weather Treatment at SEWPCC....................................11.3
11.2.2 SEWPCC Secondary Process Upgrade Options....................................11.3
11.2.3 Wastewater Characteristics....................................11.5
11.3 WET WEATHER TREATMENT ALTERNATIVES....................................11.6
11.3.1 Chemically Enhanced Primary Treatment (CEPT)....................................11.7
11.3.2 High Rate Clarification PROCESSES....................................11.9
11.3.2.1 Lamella Plate Clarification....................................11.10
11.3.2.2 Ballasted Flocculation....................................11.12
11.3.2.3 Dense Sludge Process....................................11.13
11.3.3 Retention Treatment Basin (RTB)....................................11.15
11.4 DISCUSSION AND PROCESS SHORT-LISTING....................................11.17
11.5 EXPERIENCE ELSEWHERE....................................11.20
11.5.1 Gold Bar WWTP Enhanced Primary Treatment, Edmonton....................................11.20
11.5.2 Bay View WWTP, Toledo, Ohio – High Rate Dense Sludge Process....................................11.22
11.5.3 Village Creek WWTP - Fort Worth, Texas....................................11.23
11.5.4 Willow Lake WPCF – Salem, Oregon - Peak Excess Flow Treatment Facility (PEFTF)....................................11.24
11.5.5 Lawrence WWTP, Lawrence, Kansas - Excess Flow Treatment Facility (EFTF)....................................11.25
11.5.6 Baton Rouge WWTP – Baton Rouge, Louisiana....................................11.27
11.5.7 San Francisco, California....................................11.28
11.5.8 Windsor Retention Treatment Basin ESR and Functional Design/Pilot Plant....................................11.29
11.6 PRELIMINARY JAR TESTING....................................11.30
11.6.1 Summary of Jar Test Results and Discussions....................................11.33
11.7 CONCLUSIONS....................................11.34
11.8 RECOMMENDATIONS....................................11.35
11.9 FUTURE DIRECTIONS....................................11.35

11.0 Wet Weather Treatment

11.1 INTRODUCTION

Historically, the SEWPCC experiences the highest flows during spring thaw and summer rainfall events. This is evident from Figure 11.1 that illustrates the daily flows received at the SEWPCC from 2002 to 2005. As per current operating practice, flows in excess of 100 ML/d are bypassed around the secondary process and UV disinfection while the primary treatment handles up to 174 ML/d. Based on data presented in Figure 11.1, it is evident that the secondary process was bypassed many times including bypass of primary treatment on some occasions, thereby, compromising the final effluent quality. Blending of effluents at the WWTP during periods of high flow associated with wet weather events is a common practice to protect the biological treatment process and in preventing overflows and backups elsewhere in the system (Payne, 2005). Bypass of wastewater flow around secondary followed by blending downstream of the UV disinfection may not be acceptable to Manitoba Conservation although specific directions are not provided in the new licence issued for the SEWPCC. Other jurisdictions, such as the U.S. EPA, which had made an announcement in November 2003 on the proposed policy of blending (68 Federal Register 63042-64052), have yet to finalize and implement this policy.

Treatment of wet weather flows resulting from inflow and infiltration (I/I) to the sewer collection system (as experienced at the SEWPCC catchment) is quite different from treatment of base flow during a dry weather period. For the SEWPCC, a snowmelt induced high flow event during spring when flows greatly increase the normal diurnal peak, can last for several days. Although the magnitude and duration of these events can be somewhat predicted through knowledge of the past occurrences and collection system limitations, the time of occurrence of these peak events cannot be known.

Although regulatory requirements pertaining to CSOs and sanitary sewer overflows (SSOs) continue to change, in Manitoba, specific guidelines on the reduction in frequency of SSOs and combined sewer overflows (CSOs) are still evolving. To accomplish these goals, many communities often rely on a variety of wet weather strategies that include clarification; constructing additional plant capacity; use of in-line or off-line wet weather storage; reducing peak flows through reduction of rainfall derived I/I, sewer separation, shedding/treatment of flows upstream of the WWTP or rerouting peak flows to a different treatment plant.

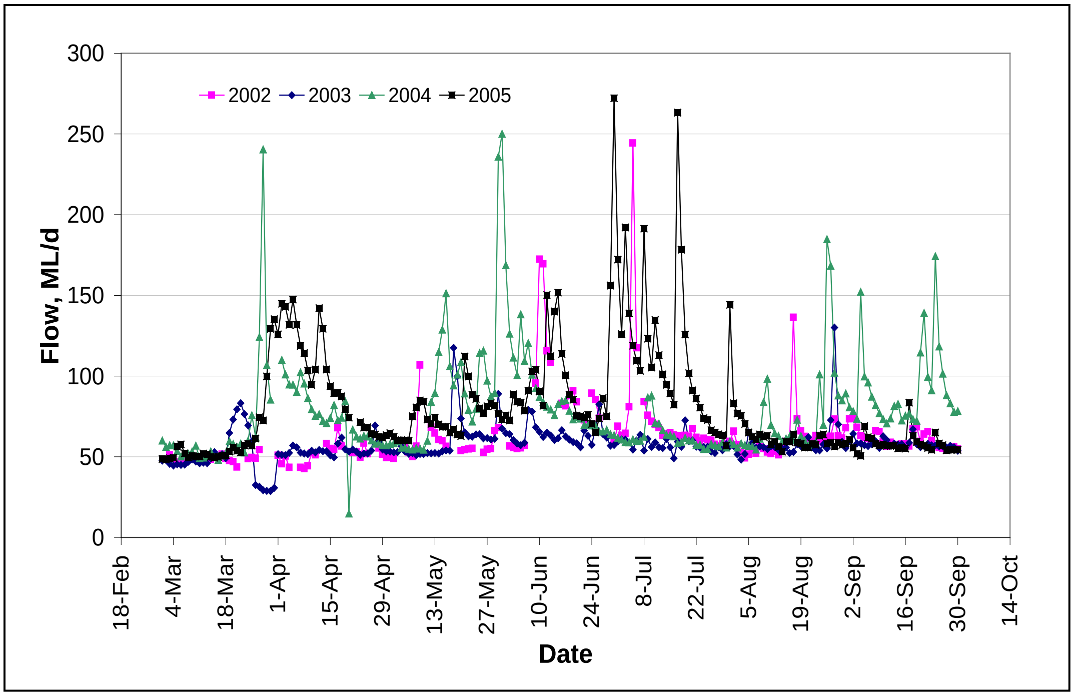


Figure 11.1 - Spring and Summer Flows Received at the SEWPCC

As seen from Figure 11.1 and historical records for the SEWPCC facility, there is a need to provide some degree of treatment to all the flows conveyed to the SEWPCC regardless of the magnitude and duration. The strategies and feasible alternatives to control the magnitude of such wet weather events reaching the SEWPCC facility is discussed in Section 10 - Wet Weather Flow Options. High wet weather flows through a rainfall and snowmelt induced inflow/infiltration can cause operational problems at a BNR facility by reducing process retention times and potentially washing out the biomass. This may result in compromised treatment efficiency for several days and potentially weeks following an event. In addition, the dilute nature of wastewaters resulting from these events is potentially more difficult to treat biologically. The potential situation at the SEWPCC presents an opportunity for the City to reduce the size of the BNR process and divert part of the flow through a side-stream treatment process, producing a final effluent that is still within the effluent limits.

There are two important reasons the City should consider the use of side-stream treatment at the SEWPCC. Since the SEWPCC experiences very high wet weather flows relative to the average day flow, implementing a side-stream treatment process for these peak flows would protect the biological process from washout of the viable biomass, thereby maintaining optimal performance of the BNR process under such conditions. The quality of the treatment plant effluent can be restored immediately after the storm event. Secondly, the cost for a side-stream treatment system is approximately one-third the cost of a BNR system and the entire treatment facilities need not be oversized to handle these unusual flow events. This could result in significant capital and operating cost savings for the City.

Although technologies such as vortex separation and high-rate compressed media filtration technologies are available, clarification is often a key component to such wet weather treatment strategies. Clarifiers used for wet weather flow treatment at a wastewater treatment plant can be storage basins operated as a flow-through system (once the basin is full), traditional clarifiers at regular loading rates, or enhanced high-rate clarification though modifications on its method of operation to increase the loading rate (and hence reduce the footprint requirement) and contaminant removal efficiency. These options are discussed in details in Section 11.3.

11.2 BASIS OF ANALYSIS

11.2.1 Maximum Day Flows for Wet Weather Treatment at SEWPCC

Section 4 presented earlier provides the basis for the population and flow projections. The approach taken assumed wet-weather Inflow and Infiltration (I/I) accounts for 25% of the existing unit area I/I, a population density of 27.5 people per hectare for the SEWPCC, per capita flows of 298 litres per capita per day (lcpd) during dry weather and 1205 lcpd under wet weather conditions. The conclusion of this analysis is that the 1:5 year summer storm event corresponds to a maximum day flow of 300 ML/d with a peak hour flow during the same event estimated at 480 ML/d.

To eliminate the requirement to transport the entire 480 ML/d to the SEWPCC, feasible alternatives were presented in Section 10 - Wet Weather Flow Options. This includes the construction of a Retention Treatment Basin (RTB) providing chemical enhanced primary treatment (CEPT) adjacent to the D’Arcy Pumping Station. The RTB would be sized to handle a 1 in 5 year Max Hour Flow at D’Arcy, which is approximately 170 ML/d.

Providing primary treatment at the D’Arcy Pumping Station and only having 310 ML/d being conveyed and treated at the SEWPCC is more economical than conveying the entire 480 ML/d wet weather flow to the SEWPCC for treatment. This approach also eliminates the requirement by the City to install a relief interceptor. The present South End interceptor on St. Mary’s Road is capable of conveying approximately 280 ML/d under open channel flow and approximately 300 ML/d with minimal surcharging but no overflow at the St. Mary’s interceptor overflow. Based on this conclusion, the analysis for the wet weather treatment options for the SEWPCC would be based on a maximum day design flow of 300 ML/d during wet weather flow events.

11.2.2 SEWPCC Secondary Process Upgrade Options

Based on a long-list of options presented in Section 8 - BNR Process Options and discussions that followed with the City during the Technical Workshop # 1, the following options for the upgrade/expansion of the SEWPCC were short-listed for further study.

Option B – High rate activated sludge process based on Modified Johannesburg configuration for biological nutrient removal (BNR) with no side-stream treatment

This option is based on a high rate BNR process that is sized to handle a maximum month spring flow (111 ML/d) and loads typical of low temperature associated with snowmelt and spring storm flow conditions. This option is capable of handling flows of 167 ML/d on a year round basis and to a maximum of 230 ML/d during the summer months when peak day flows are typically expected. No side-stream treatment is considered for this option. Flows of up to 300 ML/d would receive primary treatment with flows in excess of 167 ML/d (spring conditions) and 230 ML/d (summer conditions) by-passing the secondary process but undergoing chlorination/dechlorination prior to being discharged to the Red River. This option does not involve any dedicated side-stream treatment process for the purpose of this report.

Option C – High rate activated sludge process based on Modified Johannesburg configuration for BNR with side-stream treatment

This option is similar to Option B presented before; however, a smaller BNR process of capacity 83 ML/d is proposed (approximately 75% of the maximum month springtime flow) in conjunction with a side-stream treatment process during wet weather flows. By blending the BNR and side-stream effluents, the final effluent targets can be achieved on a 30-day rolling average basis. Capital and operating cost savings are expected to justify this strategy.

The secondary process of this BNR option is capable of handling flows of 125 ML/d on a year round basis and to a maximum of 175 ML/d during the summer months when peak day flows are typically expected. All flows up to 300 ML/d will receive primary treatment (utilizing existing conventional primary clarification + proposed side-stream clarification). The secondary process will be designed to handle maximum day flows of up to 125 ML/d (spring conditions) and 175 ML/d (summer conditions). Flows received at the SEWPCC in excess of 300 ML/d will by-pass both primary and secondary but will undergo screening and grit removal

*The side-stream treatment design will be based on a maximum flow of **125 ML/d.***

Option D – High rate HPO activated sludge process based on Modified Johannesburg configuration for BNR with side-stream treatment

This option is similar in size and concept to Option C, except that high purity oxygen (HPO) is utilized for oxygenation (compared to air in Options B and C) reducing the aerobic bioreactor cell size. This option is very compatible with the existing plant and makes use of the existing pressure swing oxygen generation facility. Similar to Option C, the secondary process of this option is capable of handling flows of 125 ML/d on a year-round basis and to a maximum of 175 ML/d during the summer months or peak day flow events.

*The side-stream treatment design will be based on a maximum flow of **125 ML/d**.*

Option I – High rate activated sludge process based on Modified Johannesburg configuration for BNR operated parallel with existing HPO plant

This option utilizes a smaller BNR process of capacity 69 ML/d in conjunction with the existing HPO plant operated in parallel. Blending the BNR and existing HPO plant effluents, the final effluent targets are achieved. The existing HPO plant essentially serves as the side-stream treatment module in this case. This option does not involve any dedicated side-stream treatment process for the purpose of this section.

11.2.3 Wastewater Characteristics

The influent wastewater characteristics and the resulting mass loads of key contaminants received at a WWTP during a wet weather flow event will be significantly different from the normal dry weather flow conditions. Although the presence of I/I usually means the measured concentrations of most constituents will be lower, significantly higher mass loads of contaminants such as suspended solids often occur during the initial “first flush” of wet weather flow event. This is normally most prevalent after a long dry period. High and prolonged wet weather flows can re-suspend sediments that may have been deposited in the collection system or scour biomass from pipe walls and transport it to the WWTP (WEF, MOP FD-8). Additionally, the characteristics of contaminants during a wet weather flow event can be very different from a dry weather flow regime. These include proportion of soluble/particulate fraction of BOD_5, TSS, fraction of particulates that can be removed by gravity settling, amounts of organic matter, frequency distributions of particle size and solids settling velocity and changes in temperature.

A detailed discussion on the wastewater characteristics with reference to SEWPCC is provided in Section 5 - Influent Characterization and Load Projections. Key observations made in the above memorandum that have relevance to implementing a side-stream treatment process are summarized as follows:

- The data collected to date, as part of the sampling and analysis program, has been relatively consistent. However, the current sampling protocol at the SEWPCC (i.e. sampling 25 mL volume for each 0.4 mL pumped) is unlikely to completely capture the larger solids peaks entering the SEWPCC during high flow events when previously settled solids are scoured from the sewer system.
- Historical primary effluent data (1995 to 2005) for the SEWPCC indicates that bioreactor influent concentration declines with increasing influent flow.
- A plot of the influent flow and TSS loading between March 1 and September 10, 2006 is presented here as Figure 11.2. This figure indicated, not surprisingly, that influent solids loadings are higher during higher flows. This suggests additional solids enter the system with infiltration/inflow and/or solids settle during lower flows and are flushed from the system with increased flows.

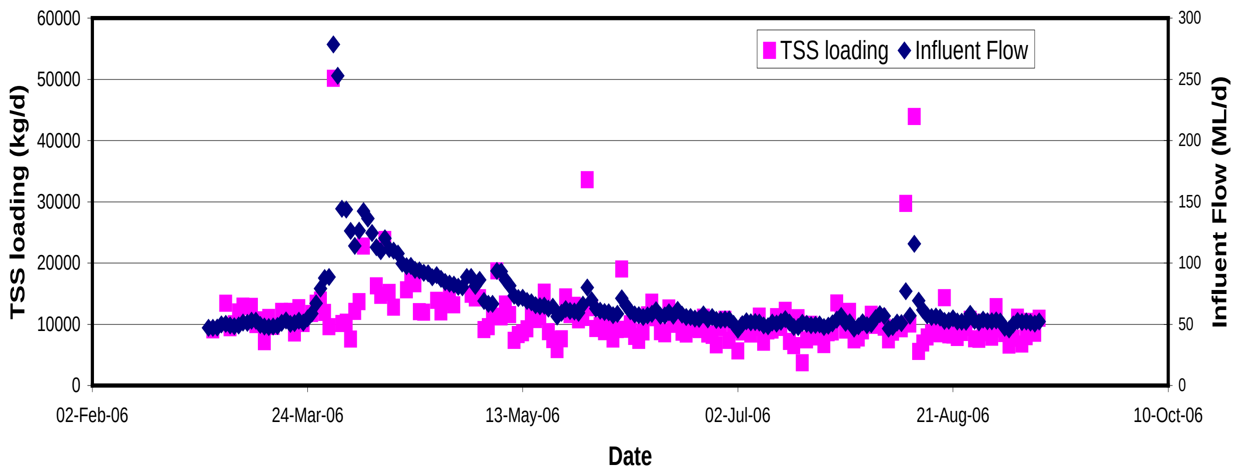


Figure 11.2 - TSS Loading and Influent Flow vs. Time (March to September 2006)

- There is a lack of data representing the characteristics of solids during a wet weather flow event. Additional sampling should be initiated to characterize influent loadings during both spring runoff and summer rainfall induced high flows in 2007.

11.3 WET WEATHER TREATMENT ALTERNATIVES

As discussed earlier, the SEWPCC currently bypasses primary effluent in excess of 100 ML/d and pre-screened/de-gritted sewage in excess of 174 ML/d. The City is interested in reducing the number of bypass events during wet weather flow events that would result in violation of the permit limits for key contaminants such as BOD_5, TSS, TN and TP. A side-stream treatment of the wet weather flows has been proposed in conjunction with two of the four BNR process trains for the SEWPCC upgrade/expansion project and the rationale for this concept was discussed earlier.

A key to the selection of an appropriate wet weather treatment process requires careful consideration of the following factors.

- the nature of the wet weather flows at the SEWPCC facility is due to high I/I (versus a CSO event) during the spring and summer months.

- wet weather treatment is required only for a short duration of time compared to the operation of the overall main plant. Since significant capital investment is required, the feasibility of the selected side-stream process to operate under normal flows should be considered.

- ability to respond to a quick start-up in response to wet weather events reaching the plant.

- ease of operation and maintenance.

- track record of similar technologies.
- costs of associated infrastructure such as building envelope requirements (inside a covered building vs. covered tanks), building footprint etc.
- capital cost and annual operation and maintenance costs.

Based on this, the wet weather treatment alternatives that are considered appropriate for the SEWPCC are listed below.

- Chemically Enhanced Primary Treatment (CEPT)
- High Rate Clarification (lamella plates, ballasted flocculation and dense sludge processes)
- Retention Treatment Basin (RTB)

Other processes such as vortex solid separators (VSS) and compressed media filtration (CMF) were not considered. The City for example has previously investigated the feasibility of the VSS technology. Based on a treatability evaluation of the Aubrey District CSO, it was concluded that VSS technology was unsuitable for Winnipeg (Wardrop/Tetres – CSO Management Study, 2002), although the treatability was on a CSO type wastewater as compared to an I/I situation for the SEWPCC sewer shed.

Very limited operating experience exists for the CMF technology. The process requires no chemical addition and as the name suggests, it operates as a filter to accomplish removal of contaminants from wastewater. Extensive piloting of the CMF technology was carried out parallel with a ballasted flocculation and dense sludge processes to the by the City of Akron. The study concluded that the CMF did not provide the level of treatment comparable to the other high-rate processes (Frank and Smith, 2006).

The alternative processes considered feasible for SEWPCC are discussed in detail in the following sections.

11.3.1 Chemically Enhanced Primary Treatment (CEPT)

In simple terms, chemically enhanced primary treatment (CEPT) involves chemical coagulation of the influent wastewater to increase the efficiency and capacity of the conventional primary clarification. The additional removal efficiency is due to the improved floc structure and increased particle settling velocity thereby enhancing treatment efficiency, measured as removal of suspended solids, organic matter and nutrients (such as phosphorus) from the wastewater. In addition, the colloidal fraction of the influent BOD_5 that would otherwise not settle in a traditional clarification process tends to flocculate better and is removed from the wastewater stream.

CEPT technology can be implemented using dedicated CEPT tanks (e.g. for use during wet weather events) or by retrofitting existing conventional primary clarifiers. The use of chemical coagulants such alum, ferric and ferrous salts in conjunction with flocculation aids such as polymer allows a higher overflow rate during the peak flow events (hence minimizing the clarifier surface area) while increasing system performance.

As applicable for conventional primary clarification, the system design of CEPT is still governed by the surface overflow rates (SOR) or rise rate. Rise rate is an important consideration in the evaluation of each side stream processes as it impacts the footprint requirement of the system tanks. Published value of peak SOR for CEPT ranges from 3.0 m/h to 5.0 m/h with removal efficiencies for TSS of 60 ~85%; BOD_5 removals of 45 ~ 65% and up to 85% removal of total phosphorus (WEF, MOP No., FD-8, 2005). The Stonecutters Island WWTP, Hong Kong is the largest operating CEPT plant in the world with an average design capacity of 1700 ML/d.

Figure 11.3, shows typical ranges of TSS removal for conventional primary treatment and CEPT versus SOR.

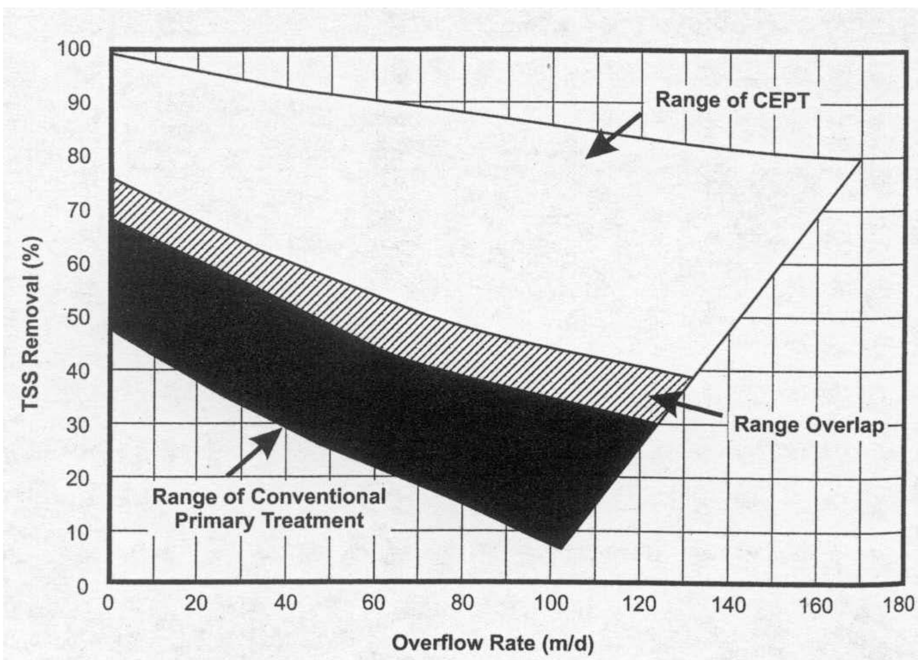


Figure 11.3 - TSS Removal with Conventional and CEPT (WEF, MOP No. FD-8, 2005)

Table 11.1 - Summary of Advantages and Disadvantages of CEPT

Advantages	Disadvantages
Increased removal of BOD, TSS, TP and metals	Requires chemical addition, which increases sludge production and increases annual operating costs.
Smaller footprint than conventional primary clarifiers	Addition of chemicals such as alum reduces alkalinity of the primary effluent causing potential impact on nitrification process
Improves performance of downstream biological process	Bigger footprint than high rate processes such as lamella plates, ballasted flocculation and dense sludge processes
CEPT tanks can be operated without chemicals during dry weather flows	More complex flow splitting and flow control as compared to conventional primary clarifiers

11.3.2 High Rate Clarification PROCESSES

Performance of all clarification devices is determined, in general, by the settling characteristics of the suspended solids i.e. settling velocity. The primary disadvantage of a conventional primary clarification process is the relatively low settling velocity of many wastewater particles which equates to a requirement for large surface areas and consequently high capital costs if they are only used for those occasional wet weather flow events.

High rate clarification processes use some combination of chemical coagulation, plate settlers such as lamella plates, ballasts/floc weighting agents or recycled sludge to achieve improved clarification performance while maintaining very high SOR. High rate clarification is very well suited for wet weather flow applications because of reduced space requirements, fast start-up, short response time, relative insensitivity to fluctuations in the influent characteristics and high degree of removal of BOD, TSS, TP, metals and TKN (WEF, MOP FD-8, 2005).

Start-up and shut down of high rate clarification in wet weather applications requires careful consideration because of their intermittent operations, the use of chemicals, and the presence of sludge and sand in the process tanks (Keller et al., 2002). Since these wet weather events cannot be predicted, polymer solutions must be made up in advance and replaced as necessary. High rate clarification processes that are used include the following: lamella plate clarification; ballasted flocculation and the dense sludge process. Further discussions on these two systems are provided in the following sections:

11.3.2.1 Lamella Plate Clarification

A further enhancement of the CEPT process can be achieved by adding Lamella plate settlers to the clarifiers, allowing operation at peak SORs of up to 12 ~15 m/h at peak conditions (HDR Engineering, Black & Veatch, 2002) and better performance than conventional CEPT. Coagulation and flocculation units are usually added upstream to enable optimum system performance. The Lamella plate clarification system uses a series of inclined plates to increase the surface area over which particles can settle out. The most significant aspect of design is its available settling area. The effective gravity settling area of the inclined plate design equals each plate's area projected on a horizontal surface. Up to ten square meters of settling area become available for each square meter of land (or floor space) occupied by the unit allowing a higher peak flow to be handled in a given tank surface area. The surface area depends upon the angle of plate inclination, which is typically around 45 to 60 degrees and spaced at intervals of 40 ~ 120 mm. Because the plates are stacked at an incline, the depth from which they must settle is significantly less than those of traditional clarifiers.

The Lamella plate clarification system has similar efficiencies as observed for CEPT. Similarly, the system can also be used for primary clarification under normal operations (without using chemicals) except that due to its unique design, influent wastewater may require pumping. Thickened sludge flows are expected to be around 2.5%. Due to incorporation of lamella plates, this option would require additional cleaning effort compared to the CEPT process discussed before. This is due to potential plugging problems due accumulation of settled solids in the plates as well as development of biofilms in the large surface area available and resulting odour generation.

There is limited application of Lamella plate clarification in North America although there are approximately 130 installations in Europe with France leading the way. The City of Edmonton Gold Bar WWTP has implemented a Lamella unit for dealing with high CSO. Some of the key design issues related to CEPT with Lamella plate includes plate settler rise rate, tank hydraulics, CFD modeling of the clarifiers under various flow regimes, end-feeding vs. side feeding, spacing between Lamella plates (minimum 75 mm recommended), and automatic plate cleaning system to avoid plugging (combination of air scour and water jets). Based on extensive piloting conducted at Gold Bar WWTP and experience elsewhere, the following key features were implemented in the final plant design:

- design rise rate of 10.2 m/hr although pilot plant showed a maximum of 14 m/hr to maintain the same effluent quality
- spacing of 100 mm in between Lamella plates
- an automatic plate cleaning system utilizing a combination of air scour and water jets

A summary of advantages and disadvantages of Lamella plate clarification systems are provided in Table 11.2. A schematic of the Lamella plate clarification is shown in Figure 11.4.

Table 11.2 - Summary of Advantages and Disadvantages of Lamella Plate Clarification

Advantages	Disadvantages
Increased removal of BOD, TSS, TP and metals	Requires chemical addition which increases sludge production and reduces alkalinity
Smaller footprint than conventional clarifiers	Scum removal can be a problem and in-place cleaning system is required to reduce clogging
Improves performance of downstream processes such as disinfection	Maintenance required for cleaning of the Lamella plates.
No additional thickening of primary sludge required	Bigger footprint than high rate processes such as ballasted flocculation and dense sludge
No additional fine screening required upstream of the CEPT clarifier	More complex flow splitting and flow control as compared to conventional primary clarifiers

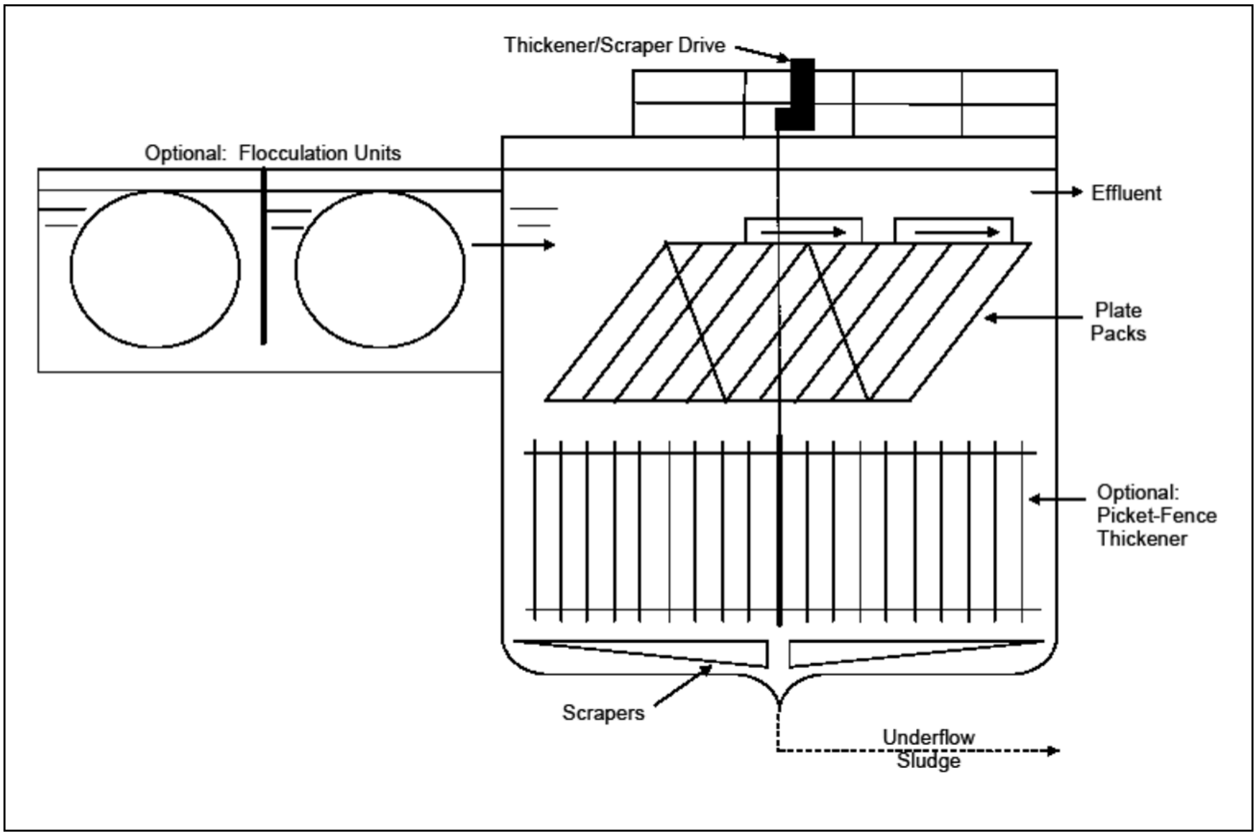


Figure 11.4 - Lamella Plate Clarification Process Schematic (U.S. EPA. 2003)

11.3.2.2 Ballasted Flocculation

Ballasted flocculation refers to a high rate clarification process that utilizes micro-sand particles (45-100 μm in diameter) to enhance floc formation and increase floc-settling rates in the presence of a chemical coagulant and polymer. This allows the system to be loaded with a very high SOR resulting in a small overall footprint. Actiflo® is the most common ballasted flocculation process used in water and wastewater applications. The system was originally developed by Kruger, Inc. now a part of Veolia Water (Cary, North Carolina) and is marketed by John Meunier (St-Laurent, Quebec) in Canada. The process schematic is shown in Figure 11.5.

Actiflo® is a three-stage process with the influent wastewater first screened and de-gritted to remove large particulates prior to entering the first-stage. The first step is usually the addition of a coagulant such as alum or ferric salts prior to flash mixing followed by the addition of polymer and micro-sand. The second stage of the Actiflo® process is maturation, where the ballast material serves to enhance the flocculation process, resulting in a much faster settling rate relative to traditional coagulants. The third stage of the Actiflo® process is clarification. A majority of the solids settles to the bottom of the tank. However, the clarification zone is equipped with Lamella plates to further enhance the solid-liquid separation process. The settled solids are recycled back to a hydrocyclone where the sludge is separated from the micro-sand. The sludge is wasted and the micro-sand is retuned back into the process in the injection zone. Typical removal efficiencies for this process range as follows: TSS (70 ~ 90%); BOD_5 (40 ~ 60%); TP (70 ~ 96%) and TKN (17 ~ 30%).

The Actiflo® process can treat flows between 10 and 100 percent of its nominal design capacity, allowing systems to provide wet weather treatment for a range of design storm events. Typical start-up to steady-state time ranges from 15 to 30 minutes (to be confirmed by pilot testing). Typical peak surface overflow rates for the Actiflo® process in the treatment of wet weather flows are in the range of 100 to 130 m/hr and produces thickened sludge in around 0.3 ~ 1% solids (HDR Engineering, Black & Veatch, 2002).

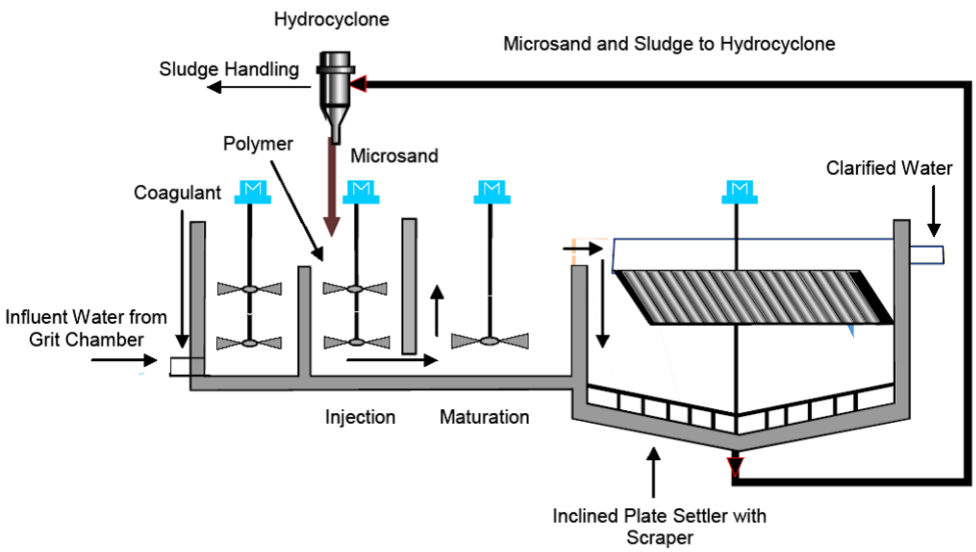


Figure 11.5 - Actiflo® Process Schematic (U.S. EPA. 2003)

Table 11.3 - Summary of Advantages and Disadvantages of Ballasted Flocculation

Advantages	Disadvantages
Smallest footprint amongst all side-stream processes considered for SEWPCC	Increased dosages of coagulants and polymers compared to CEPT
Fast start-up and shut down	Requires fine screening ahead of the process, which increases capital and operational costs.
Very high degree of TSS, BOD, TP removal	Very low sludge concentrations, requires additional thickening of sludge
Process maintains stability even at high SORs	Micro-sand management issues during start-up and shut-down and higher wear rates for pumps and piping moving sludge and sand

11.3.2.3 Dense Sludge Process

Dense sludge is a high rate clarification process that combines chemical coagulation, sludge recirculation, tube settling, thickening, and sludge recycling. Unlike the use of micro-sand in the ballasted flocculation process, a portion of the settled sludge (2 to 6% of flow) is recycled to the bottom of the flocculation tank resulting in a dense floc with high settling velocities. This technique allows for high removal efficiencies of TSS, particulate BOD and TP even under very high SORs. The dense sludge process is marketed under the trade name of DensaDeg® by Infilco Degremont, Inc. (Richmond, Virginia). A schematic of the process is shown in Figure 11.6.

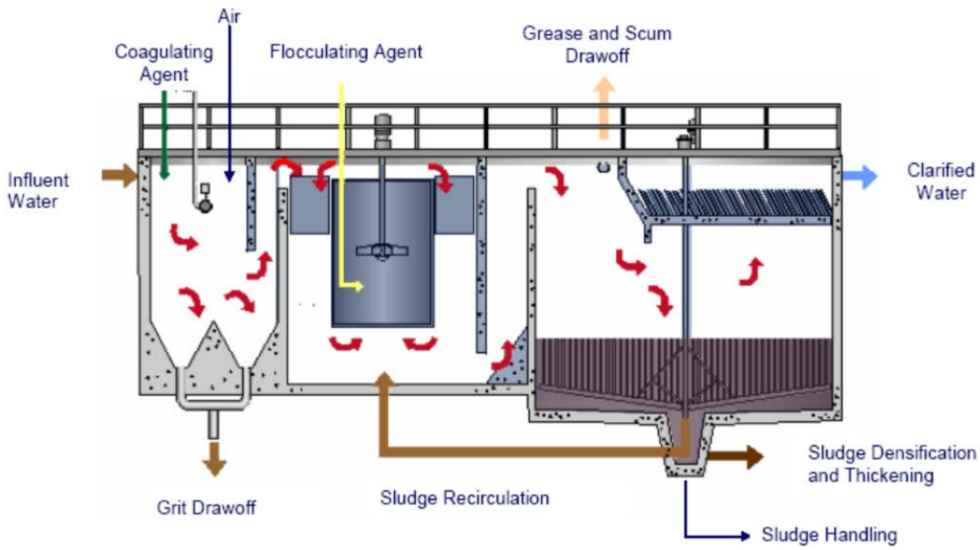


Figure 11.6 - DensaDeg® Process Schematic (U.S. EPA 2003)

The DensaDeg® process is capable of the rapid start-up and shutdown which will typically be required for responding to wet weather flow situations experienced at the SEWPCC. When this process is started dry, full efficiency is attained within 20 ~ 30 minutes and almost immediately during wet start-up (Westrelin and Bourdelot, 2001). Some of the unique features of the DensaDeg® process are the use of air injection simultaneously with the coagulant and the use of a draft-tube mixer to enhance coagulant dispersion and mixing.

Coagulated wastewater enters the reactor where polymer is added with recycled settled sludge to help the flocculation process. In the reaction zone, wastewater enters a clarifier where grease and scum are drawn off the top. In the final step of the process, inclined tube settling or lamella plate settlers are used to remove residual floc particles. Settled sludge from the clarifier is thickened, and part of this sludge is re-circulated and added to the flocculate. Because this system uses entirely recycled sludge as a coagulant aid, it does not require separation techniques such as the hydro-cyclone in Actiflo® system to recover micro-sand from the sludge. Typical peak surface overflow rates for the DensaDeg® process are in the range of 30 to 100 m/hr (HDR Engineering, Black&Veatch, 2002) with thickened sludge concentrations of approximately 4% solids or 40,000 mg/L. i.e. producing sludge almost 4 to 13 times thicker than Actiflo®. This difference in sludge concentration is one of the important aspects for selecting an appropriate side-stream process for SEWPCC. Since the DensaDeg® and the Actiflo® process are expected to produce a similar mass of sludge (as they operate with similar coagulant dosages), the volume of sludge produced by DensaDeg® process would be significantly less than the Actiflo® process. Since sludge is hauled from SEWPCC to the NEWPCC, this represents an additional cost to the City, which should be factored in the final selection of the side-stream process. The treatment efficiencies of this process for the key contaminants such as BOD_5, TSS, TKN and TP are comparable to the Actiflo® process, although at comparatively lower SORs.

Table 11.4 - Summary of Advantages and Disadvantages of Dense Sludge Process

Advantages	Disadvantages
Footprint smaller than Lamella plate clarifier but slightly larger than Actiflo® system	Requires fine screening ahead of the process
Produces sludge with highest concentration of solids that equates to lowest volume of sludge. No additional thickening of sludge is required	Requires longer time needed for startup because of the time required to build up re-circulating sludge from influent TSS
Very high degree of TSS, BOD, TP and TN removal (similar to Actiflo®)	Potential for septic conditions and resulting in odors and corrosion if sludge is not properly managed in between start-up and shut down operations

11.3.3 Retention Treatment Basin (RTB)

A Retention Treatment Basin (RTB) consists of wet weather flow storage tank or a vessel that provides some storage and treatment in a flow-through mode (Schraa et al., 2004). A typical RTB can resemble both a storage tank and clarifier and can operate in conjunction with chemical coagulation for enhanced treatment. During the flow-through treatment, influent solids are captured from the wastewater as settled sludge and floatable materials are removed. Both sludge and floating solids are typically returned to the mainstream process for further treatment/handling.

After a wet weather event has ended, the draining and flushing systems provide for draining the stored wet weather flow in the RTB to the outfall or interceptor sewer and for flushing out settled solids. Solids and flushing water are also discharged to the outfall pipe/interceptor sewer. Hence RTBs have flushing systems rather than sludge scrapers for diverting the solids back to the main treatment plant.

Rectangular basins are preferred as they are least expensive to construct and maintain. Baffles are generally used as a part of the inlet designs to reduce inlet velocity and promote plug flow conditions to maximize sedimentation efficiency. Outlet structure design is critical to maintain a constant outlet flow rate to the downstream processes or structures. Fixed outlet orifices, flow restricting pipes, and overflow weirs are often chosen because they have predictable hydraulic characteristics and are simpler to design (U.S. EPA Fact Sheet, 1999). Disinfection is provided by the retention and chlorination of the influent to the RTB structure. Dechlorination is recommended to the portion of effluent discharged to a surface water body.

The sizing and capacity of RTBs are dependent on three key principles:

- The hydraulic characteristics of the wet weather flow to be treated, including volume and peak flow distribution;
- The characteristics of the settleable solids in the overflow and the fraction of suspended solids that are non-settleable; and
- The required performance of the settling basin in terms of either percentage removal or effluent concentration.

An example of a RTB concept for CSO control at Lou Romano Water Reclamation Plant (LRWRP) in Windsor, Ontario is shown in Figure 11.7.

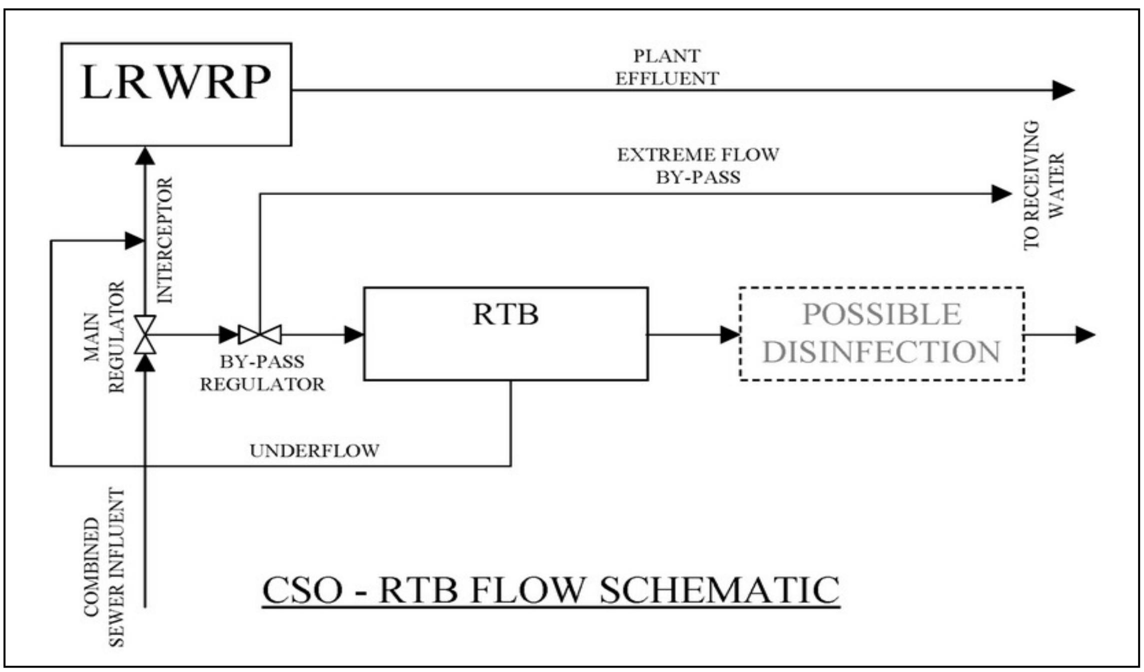


Figure 11.7 - Typical Layout of a Retention Treatment Basin (Source: Stantec, Windsor)

Table 11.5 - Summary of Advantages and Disadvantages of RTBs

Advantages	Disadvantages
Simple operation compared to other side-stream processes	Largest footprint amongst all side-stream process
No additional fine screening required upstream of RTB	Potential for septic conditions and odour problems
No major mechanical parts	Contaminant removal efficiency is lower than that of other high rate clarification processes

11.4 DISCUSSION AND PROCESS SHORT-LISTING

The key issues in the short-listing of an appropriate wet weather flow treatment process for the SEWCC is based on several factors including: water quality objectives; overall value of the process with respect to the City's operational goals, process flexibility; ease of operation and land area requirements.

The experience with vortex type separators for treatment of CSO type wastewater has not been positive for the City in the past. Similarly, there is limited operating experience with compressed media filtration in such a large scale as proposed for the SEWPCC. As such, these two technologies were not considered further for SEWPCC. RTBs have been implemented in several locations as remote or a satellite type facility primarily for CSO applications. As stated in Section 3.4, RTBs presents challenges with solids handling, odour potential and has the largest footprint. It is likely that Manitoba Conservation will require the basin design to follow the similar guidelines as in sewage lagoons. In absence of clay, the RTB cell may have to be lined for SEWPCC. Based on these discussions, the high rate clarification option is short-listed for further considerations as side stream treatment for SEWPCC. These options include the following processes:

- lamella plate clarification
- ballasted flocculation (Actiflo®)
- dense sludge process (DensaDeg®)

There is limited operating experience with Lamella plate clarification in North America. Two of the largest facilities include Longueil WWTP near Montreal, Quebec and the Gold Bar WWTP, Edmonton, Alberta. The Longueil lamella plate clarification system has been in operation for quite sometime whereas the Gold Bar facility is expected to be in operation by early 2007. The Lamella plate option provides the City to operate it as a high rate clarification process during peak wet weather flow events and as a regular primary clarifier (i.e. without any chemical addition) during dry weather periods. Under these conditions, the overall primary effluent can be significantly improved due to lower operating SORs and the resulting primary sludge will have higher solids content because of the higher retention time in the clarifiers.

Given the limited number of wet weather events expected each year and the capital expenditure required for this option, the year round utilization of Lamella plate clarifiers maximizes the utilization of process resources. In addition, the Lamella plate clarifiers could serve as a back up to the existing primary clarifiers should maintenance is required. Piloting is however, strongly recommended to evaluate performance requirements and to optimize key process parameters during wet weather events at the SEWPCC. The pilot plant constructed for the Gold Bar WWTP is available for use.

As a second option to lamella plate clarification, either the Actiflo®) or the DensaDeg® process can be short-listed for side-stream treatment at SEWPCC. Both technologies offer similar removal efficiencies for TSS, BOD_5 and TP, however the Actiflo®) process offers some additional benefits. The process can achieve the highest surface overflow rate amongst all technologies currently applicable (ranges from 100 to 130 m/h). This allows for a smaller footprint and shorter retention time, enabling a faster start-up time and recovery time from failure compared to the DensaDeg® process. The longer start-up time required for the DensaDeg® process is because of the time required to build up re-circulating sludge from influent TSS. Should the DensaDeg® process tanks be allowed to hold sludge for a long period of time in between wet weather events, there is a great potential for septicity of the re-circulating sludge. In the recent years, the Actiflo®) based ballasted flocculation technology has been successfully used in North America for side-stream wet weather treatment (more widely than the DensaDeg® process) with over a dozen facilities currently in operation and several others in design or construction stages.

Both Actiflo®) and the DensaDeg® are patented processes that rely heavily on chemicals (coagulant and polymer) in conjunction with ballasts (sand) or re-circulating sludge. With both processes, either the loss of chemical feeds or loss of sand ballasts/stoppage of sludge recirculation, results in significant loss in treatment efficiency. Pilot studies have also indicated the Actiflo® process achieves similar rates of removal as the DensaDeg® process, with lower chemical dosages. On the other hand, the DensaDeg® process produces a smaller volume of sludge with a higher percentage solids concentration than Actiflo®. The DensaDeg® process also does not require fine screening (6 mm or less) ahead of the treatment tanks which increases the overall capital and operating costs for the Actiflo® option.

Because such high-rate clarification process may only be used for a few times a year, there is an opportunity to use this process during dry weather flows for tertiary treatment for TP and TSS removal. This may be a benefit to the City in the future when tighter effluent TP limits are anticipated. Pilot plant experiments were conducted by Stantec at the Regional Wastewater Treatment Plant (WWTP), located in the Town of New Tecumseth, Ontario. The WWTP is required to meet a stringent future effluent total phosphorus limit concentration of less than 0.07 mg/L. This total phosphorus limit is typically not achieved using traditional forms of phosphorus removal in an activated sludge system followed by tertiary filtration. As a result, the Actiflo®) and the DensaDeg® processes were pilot tested to confirm their ability to meet the required phosphorus limits. The Actiflo® process consistently produced effluent total phosphorus concentrations less than the test target of 0.07 mg/L. The DensaDeg® pilot process experienced floating sludge problems, believed to be caused by a large industrial component to the influent wastewater at the Regional WWTP. As a result, the DensaDeg® process did not meet the treatment objective of less than 0.07 mg/L effluent total phosphorus.

Based on the discussions presented and the relative advantages disadvantages of each of the high-rate clarification processes, both the lamella plate and the Actiflo® technology are short listed for pilot studies and further considerations. A concept utilizing the Lamella plate clarifier and the Actiflo® technology for side-stream treatment at SEWPCC (for Options C and D only) is shown in Figure 11.8 and Figure 11.9 respectively.

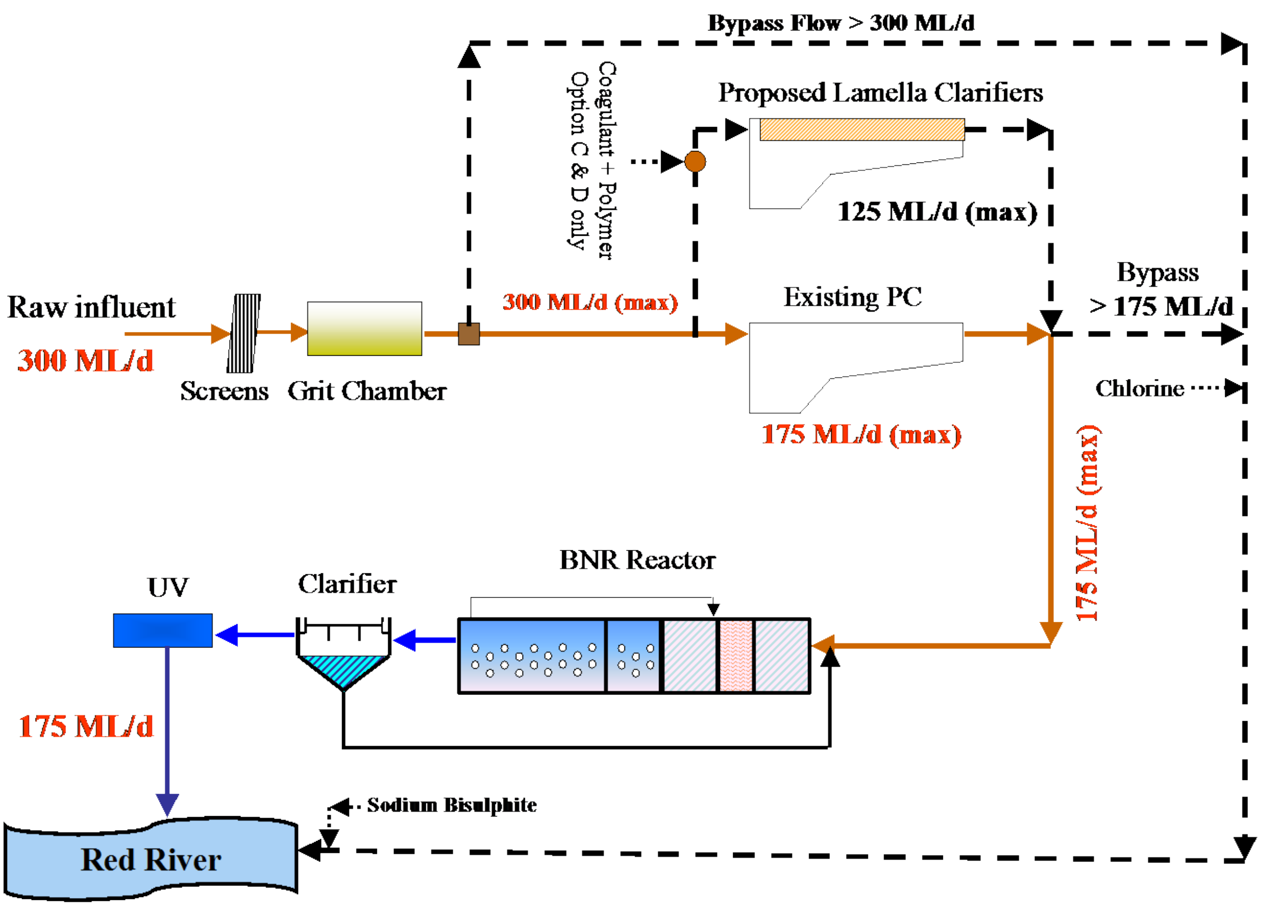


Figure 11.8 - SEWPCC Side-Stream Option Based on EPT with Lamella Clarifier

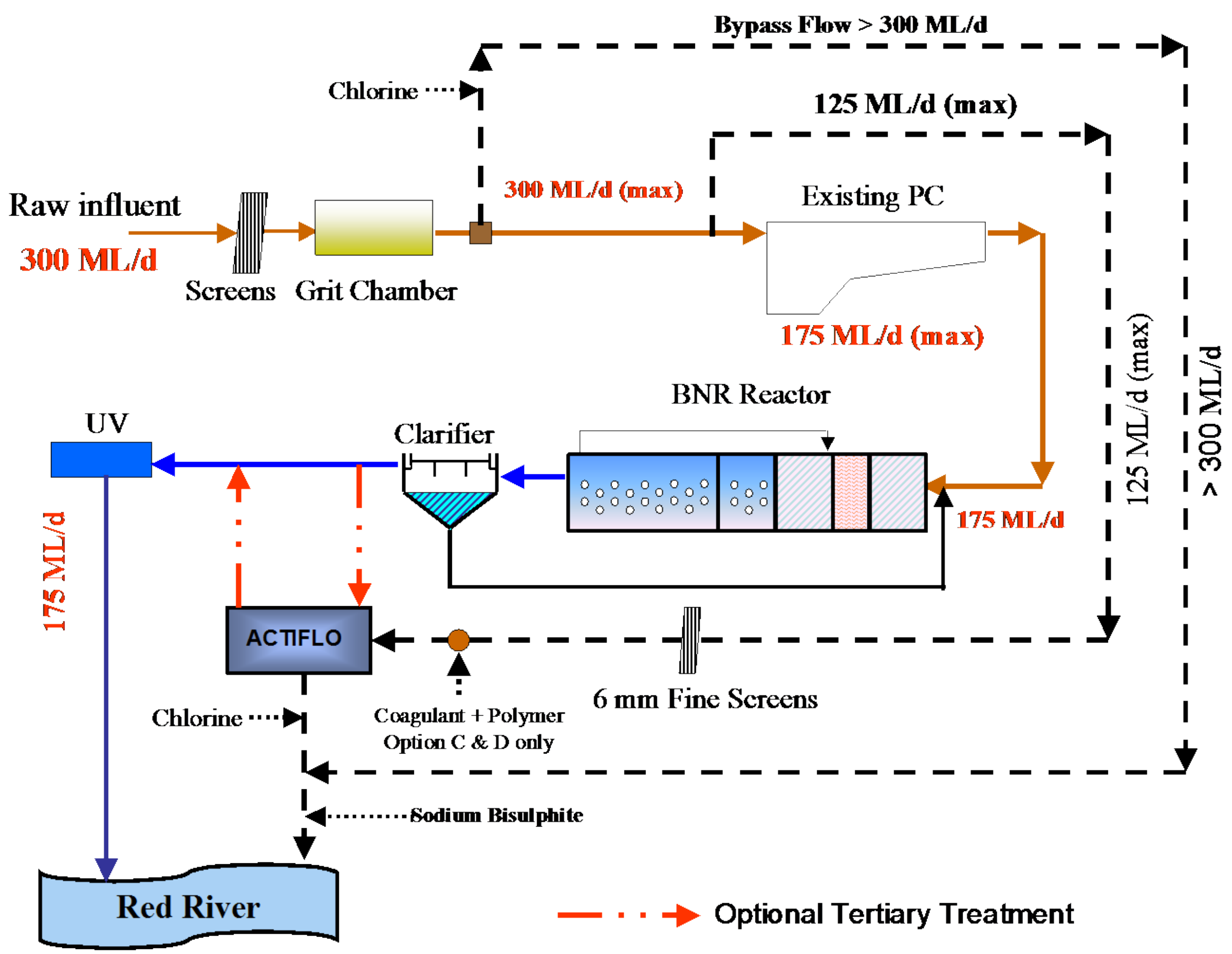


Figure 11.9 - SEWPCC Side-Stream Option Based on Ballasted Flocculation (Actiflo®)

11.5 EXPERIENCE ELSEWHERE

This section provides an overview and discussions on the application of alternatives wet weather treatment processes in Canada and the USA.

11.5.1 Gold Bar WWTP Enhanced Primary Treatment, Edmonton

As a part of the CSO Long Term Control Plan, the City of Edmonton implemented a plan to upgrade the Gold Bar Wastewater Treatment Plant (GBWWTP) primary treatment facilities to reduce CSO discharges to the North Saskatchewan River. The City commissioned a study in 2000 to review and evaluate alternative disinfection and enhanced primary treatment alternatives as they apply to CSO treatment at the plant. During this study, the following treatment technologies were evaluated:

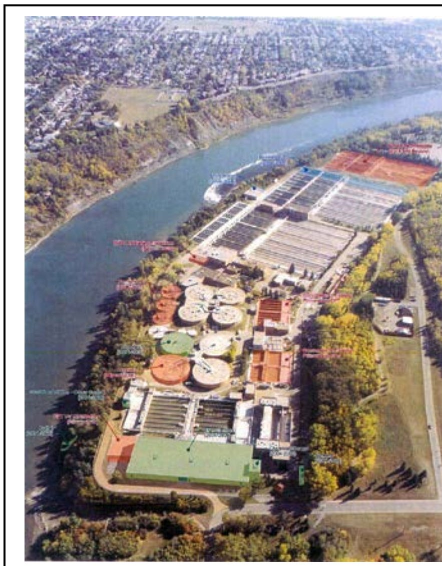
- Conventional enhanced primary treatment (CEPT);
- Lamella enhanced primary treatment;
- High rate enhanced primary treatment (including Lamella clarifier, Actiflo® and DensaDeg® etc.; and
- Disinfection (including hypochlorination – dechlorination, ozonation, ultraviolet irradiation).

Actiflo® (ballasted flocculation) was ultimately recommended as the preferred technology for enhanced primary treatment (EPT) at GBWWTP followed by ultraviolet (UV) irradiation was recommended as the preferred technology for EPT effluent disinfection.

In the Spring 2002, the City of Edmonton commissioned a pilot study to further evaluate the following technologies:

- EPT technologies (including Actiflo®, CEPT, chemically enhanced Lamella plate settling; and
- Disinfection technologies (including hypochlorination – dechlorination, UV irradiation).

The pilot studies determined that both Actiflo® and Lamella plate settler effluent quality was sufficient for UV disinfection to effectively meet the fecal coliform design target of 1000 cfu/100 ml. On the basis of results of the pilot testing program, as well as associated system costs, Actiflo® technology and UV disinfection was again recommended. Subsequent to this recommendation, an independent team further evaluated the test data including a Value Engineering exercise and determined that Lamella plate clarification followed by UV disinfection is more suitable for the GBWWTP (Stantec, 2003). The decision was based on the fact that the EPT produces a much thicker sludge, did not require additional fine screening on the upstream or additional maintenance associated with sludge/sand recycle pumps and hydrocyclone operations. Additionally, space was not an issue at Gold Bar for the construction of the proposed EPT clarifiers. In addition, the City realized that the EPT clarifiers provided a value added alternative by being available during the non CSO events to improve the overall treatment efficiency of the WWTP and also serving as stand-by units to the existing conventional primary clarifiers.



With the EPT technology defined, the City of Edmonton has proceeded with implementation of the necessary facilities. The key components of the newly completed EPT facility are summarized as follows:

- **Enhanced Primary Treatment (EPT) Clarifiers** – 600 MLD capacity chemically aided clarification with inclined plates for enhanced removal of suspended solids and other pollutants from the wastewater. There are four (4) EPT clarifiers, each with 150 ML/d capacity (see attached photo showing rendering of the EPT clarifiers in green in foreground)
- **Screens** – 1000 MLD capacity screens for the removal of floatables and from the wastewater. These facilities will be utilized only during extreme wet weather events, when the capacity of the plants primary, secondary, and EPT clarifiers is exceeded.
- **Chemical Building** – Chemical storage facilities are required to store and feed the treatment chemicals (alum and polymer) to the EPT process.
- **Odour Control Facilities** – Design target is zero increase in plan odour emissions from the EPT process.
- **Conduits and Outfall** – Conveyance facilities are required to convey screened wastewater around the EPT clarifiers to the river outfall, as well as convey wastewater past the screen.

The start-up and commissioning is expected in early 2007.

11.5.2 Bay View WWTP, Toledo, Ohio – High Rate Dense Sludge Process

Located near the mouth of the Maumee River, the Bay View WWTP is one of the largest wastewater treatment facilities in northwest Ohio. Owned by the City of Toledo, it also serves other areas including the City of Rossford, the Villages of Walbridge and Ottawa Hills, and portions of Wood County, Lucas County and the Village of Northwood. The population of the service area is approximately 398,000. The Toledo area wastewater collection system is composed of combined sanitary and storm sewers in the older sections of the city, and separate sanitary sewers in the newer areas. The wastewater flow is delivered to Bay View through three main interceptors. The Bay View facility is responsible for the interceptor sewers, four large pump stations, 35 small lift stations, and 33-combined sewer overflow regulators. The four large pump stations include Bay View, East Side, Reynolds Road, and Windermere. Nine of the small lift stations are storm water stations and 26 are sanitary lift stations.

The wastewater discharged into the wastewater system is generated by three main sources: industrial, domestic/commercial and extraneous flow. Flow contributions from these sources are respectively 21%, 30%, and 49%. The extraneous flow constitutes a significant portion of the waste volume and originates from the antiquity of some sewers and the effect of the combined sewers. The treatment train is comprised of screening, grit removal, pre-aeration, primary clarification and aeration followed by conventional secondary treatments and step-aeration activated sludge processes. Effluent discharge is to the Maumee River.

The City has embarked on a plan to construct a new 227 ML equalization basin and a 600 ML/d high-rate side-stream wet weather treatment process to prevent any further discharges of untreated wastewater into the Maumee River during heavy rains. The City selected the dense sludge process based on DensaDeg® technology by Infilco Degremont, Inc. (Richmond, Virginia). The treatment process includes six (6) DensaDeg® Clarifier/Thickener units. The plant is expected to be in operation by the end of 2006. Shop fabrication of stainless steel components for six (6) DensaDeg® clarifiers are shown above.

11.5.3 Village Creek WWTP - Fort Worth, Texas

The City of Fort Worth's Village Creek Wastewater Treatment Plant is a 628 ML/d (166-MGD) activated sludge treatment process with anaerobic digestion, biosolids reuse, digester gas recovery and reuse through two 5-MW gas turbine engines. Treated effluent is discharged into the Trinity River - a sensitive stream that also receives treated wastewater from surrounding counties. During dry months, the river may at times be composed of up to 95 percent wastewater. However, during wet weather flows, the WWTP experienced flows in excess of 965 ML/d. To resolve this problem, the City needed a management strategy to control overflows from its wastewater collection and treatment system during wet-weather events, in compliance with an EPA administrative order.

Expanding the conventional treatment process to handle these high flows was estimated to cost $50 million. Initial pilot studies demonstrated the removal of more than 85 % TSS, 65 % BOD, 80-90 % TP and 20-30% nitrogen (Payne, 2005). Following these initial tests, a comprehensive pilot study was undertaken to evaluate the performance of four HRC process equipment alternatives at various overflow rates. The alternative processes included: Ballasted flocculation (Actiflo®, Microsep), dense sludge (DensaDeg®), and Lamella plate clarification. A summary of the pilot test results is as follows:

- TSS and TP removal in the order of 70 to 90% were achieved with some exceptions.
- BOD removals ranged from 35 to 65% depending on the process and overflow rate.
- Nitrogen removals were on the order of 20 to 30%.

In addition, the following observations were made (Sawey and Gerrity et al. 1999):

- Actiflo and DensaDeg® processes reached peak operating performance within 20 minutes of operation.
- Lamella clarifier unit did not reach peak efficiency until approximately 120 minutes of operation due to its longer retentions times, though the system performed effectively after 20 minutes of operation. The long start up time did not make this process feasible for peak wet weather flows.
- The results of these tests identified optimum coagulant dosages and start-up procedures for full-scale facilities and finally helped Fort Worth to obtain the first National Pollutant Discharge Elimination System (NPDES) permit to send excess influent wastewater to the high-rate process (HRC) during peak wet-weather flows. The Actiflo® system from Kruger Inc., (currently Veolia Water) was selected. The cost of high-rate process was calculated at $0.20 per gallon, compared to activated sludge process at $1.58 per gallon (based on 100 MGD facility).
- Operational costs of high-rate processes were estimated at $90 per MGD, compared to activated sludge process at $30.70 per MGD. Even though the operational costs of high-rate processes are very expensive compared to conventional activated sludge (AS) process, they are much more economical to operate infrequently, compared to the cost of conventional AS process built for peak wet weather flow and operated year round. The above photo shows the facility under construction (source: CDM).

11.5.4 Willow Lake WPCF – Salem, Oregon - Peak Excess Flow Treatment Facility (PEFTF)

The Willow Lake WWTP services an estimated population of 200,000 people treating on average 283 ML/D. During peak wet weather conditions flows can reach more than 1135 ML/D, exceeding the capacity of both conveyance (587 ML/D) and treatment systems (397 ML/D). These events cause the discharge of untreated sanitary sewer overflows (SSOs) at permitted outfalls. The City is required to eliminate the amount of SSOs by year 2010. The expansion of conventional AS process to meet the future effluent requirements, was estimated at over $400 million (Matson et. al. 2002). As a result, the City decided to explore possible alternatives to treat wet weather flows, one of which involves the use of preliminary treatment and high rate clarification (HRC) process coupled with UV disinfection.

The initial pilot tests were conducted at the Willow Lake facility over a two-year period in 2001 and 2002. The objective of the study was to demonstrate the feasibility of using HRC operating in series with UV disinfection for treatment of dilute sanitary sewer flows. Feasibility was defined by production of pilot effluent that was consistently equivalent or superior to the quality of effluent currently discharged from the Willow Lake secondary treatment.

Two HRC processes were piloted: Actiflo® and DensaDeg®. Two types of UV systems were also piloted: medium pressure (MP) and low-pressure high output (LPHO). Pilot tests results can be summarized as follows (Matson, Eckley et al., 2002):

- Both systems: Actiflo® and DensaDeg® removed 85 to 90% TSS and 50 to 70% BOD.
- Given optimized coagulation, clarifier SOR showed limited impact on Actiflo® process performance allowing to push the system past 240 m/hr for brief periods without loss of solids in clarifier. DensaDeg® system was more sensitive to high SORs and was limited to no more than approximately 98 m/hr.
- Both UV systems were able to lower *E.coli* concentrations in the effluent to below 126 per 100 mL (disinfection goal). An UV dose of 30 to 40 mJ/cm^2 was sufficient to provide required disinfection.
- A conveyance and treatment scenario that includes up to 605 ML/D of peak capacity through remote HRC system has the potential to save the City over $40 million in project costs.

The construction of the Peak Excess Flow Treatment Facility (PEFTF) based on the Actiflo® technology was subsequently undertaken by the City of Salem to treat sanitary sewer overflows (SSOs) before discharge to the Willamette River. The facility was designed and constructed in multiple phases based on projected flows associated with collection system improvements. The first phase was completed to treat peak flows of 190 ML/d (50 MGD) consisting of two 95 ML/d (25 MGD) Actiflo® units. The second phase will be constructed to treat projected peak flows of up to 455 ML/d (120 MGD) by 2010 and the final phase will be constructed at an appropriate later date to treat peak excess flows of up to 606 ML/d (160 MGD).

The PEFTF facility consists of treatment processes including pumping, 6 mm fine screening, high-rate clarification based on Actiflo® technology, and ultra-violet light disinfection. The design of the facility includes mitigation of odor, noise, and aesthetic issues, an operations area, and integration with park amenities. The facility operates fewer than ten times per year, on average. Operation events typically range in length from a period of a few hours up to one to two days.

11.5.5 Lawrence WWTP, Lawrence, Kansas - Excess Flow Treatment Facility (EFTF)

The Lawrence Wastewater Treatment Plant was recently expanded to accommodate City growth and to meet facility rehabilitation needs for the design year 2020. The project expanded the main wastewater treatment process and included an excess flow treatment facility (EFTF) to treat peak flows during storm events. The main process was designed to treat 95 ML/d (25 MGD) during wet weather events. Since the peak flows in 2020 are predicted to reach 246 ML/d (65 MGD), the EFTF was sized to handle 152 ML/d (40 MGD). Historical data of Lawrence WWTP indicated that when flows are more than double the average flow, wastewater strength is significantly reduced. During those conditions, treatment by a combination of ballasted flocculation and disinfection would allow the plant to meet effluent discharge requirements (Keller and Schultze, 2004).

Based on this approach, two (2) Actiflo® units, each capable of treating 76 ML/d (20 MGD) were chosen as a high rate clarification technology. In addition to the ballasted flocculation basins, a flow splitter—screening facility, a disinfection basin, and a chemical storage and feed facility were constructed. Excess flows to the WWTP greater than 95 ML/d (25 MGD) are pumped from the head of the plant by-passing normal treatment and through fine screens to the Actiflo® basins. Ferric chloride along with polymer is part of the chemicals added along with micro-sand to form ballasted floc, which settles quickly and is removed from the flow. The effluent from Actiflo® is chlorinated in a dedicated chlorine contact basin followed by dechlorination with sodium bisulfite. The effluent from the EFTF process is combined with the normal plant effluent and discharged to the Kansas River.

Data from the first year of operation (2003) indicated that system encountered storm events that lasted anywhere from 4 hours to 47 hours. The observed TSS removal was around 88%, accompanied by approximately 80% removal of turbidity. Chemical cost per MGD of treatment was around \$200 to \$300. Actiflo® solids are recycled to primary clarifiers and because of addition of polymer and ferric chloride primary clarification performance was enhanced. Reduction in soluble BOD load on the main activated sludge process required a change in the sludge-wasting rate to keep the process stable. MLSS concentration could decrease up to a maximum of 40% during storm events if sludge wastage rate was not adjusted. Overall, during the first year of operation, the Actiflo® system performed very well, with only one exception when TSS exceeded the limit of 45 mg/L (Keller et. al., 2005).

A schematic diagram of the Lawrence WWTP excess flow treatment facility (EFTF) is presented in Figure 11.10.

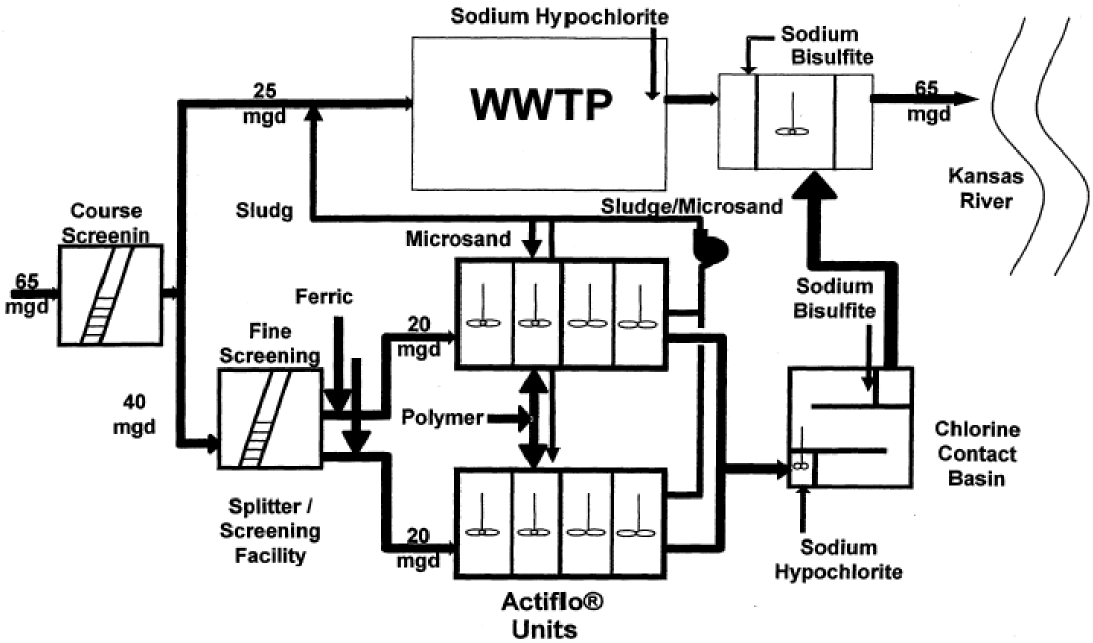


Figure 11.10 - Lawrence WWTP Liquid Process Schematic (Adapted from Keller and Kobvlinski et al. 2005)

11.5.6 Baton Rouge WWTP – Baton Rouge, Louisiana

The Baton Rouge WWTP experienced very high peak flows during wet weather events caused by high storm water infiltration and inflow. These high flows exceed the maximum treatment capacity, which results in periods of non-compliance. In February 2004 two ballasted flocculation systems: Actiflo® and DensaDeg® were pilot tested side-by-side in order to demonstrate applicability of those technologies to treat wet weather flows (Kirby et. al. 2005).

Pilot systems were operated as high-rate clarifiers during peak storm events and as tertiary clarifiers during normal flow conditions. The results of pilot testing demonstrated that both technologies were able to meet effluent criteria. During simulated wet weather runs (systems fed with raw plant influent flow during rain events) both systems exceeded the treatment goal of 45 mg/L for BOD and TSS effluent concentrations. In addition, both processes approached and at times exceeded the goal of 85% removal for both TSS and BOD. The optimum rise rate for the Actiflo® system was approximately 146 m/hr, while the optimum rise rate for DensaDeg® system was between 98 to 122 m/hr. The Actiflo® system was able to achieve stable operation within 10 minutes of operation, while the DensaDeg® system required 45 minutes from startup to achieve optimum performance. Additionally, both systems proved capable of further improving the quality of plant effluent after secondary clarification (Kirby et. al. 2005).

11.5.7 San Francisco, California

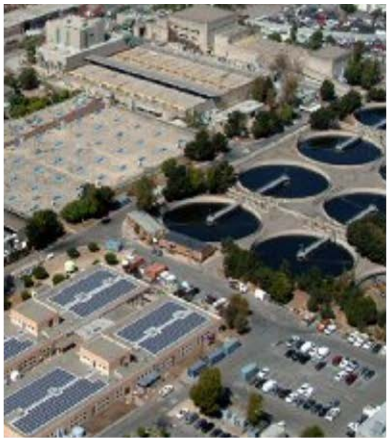
San Francisco has nearly 900 miles of sewers, three treatment plants, 36 overflow points, four outfalls, and 17 pump stations. San Francisco Public Utilities Commission (SFPUC) treats and discharges approximately 318 ML/d (84 MGD) of treated wastewater during dry weather to the San Francisco Bay and Pacific Ocean. During wet weather, with additional facilities and increased operations, the existing three (3) WWTPs can treat approximately 1760 ML/d (465 MGD) of combined flows per day. The Southeast Water Pollution Control Plant near treats sewage from the eastern side of the City and the Oceanside treats sewage from the western side. Both plants provide full secondary treatment to the majority of storms throughout the wet weather season. During wet weather events, the operators also start up the North Point Facility (NPF) to provide primary-level treatment for combined storm flows.

The current capacity of the North Point Facility (NPF) plant is the limiting factor for the efficiency of the system and the expansion of the facility is difficult due to the space constraints (Jolis and Ahmad 2001). Two high-rate clarification technologies: Actiflo® and DensaDeg® were chosen as appropriate systems for upgrade due to reduced space requirements. Very high settling velocities combined with rapid flocculation kinetics lead to plant footprints less than 10% of conventional primary treatment. The pilot plants were actually set-up and operated at the Southeast Water Pollution Control Plant (SEWPCP). The pilot program included two phases: optimization (on blended raw influent and secondary effluent during dry weather days and wet weather influent during wet weather days) and demonstration (on wet weather influent).

Steady state performance of the two processes was comparable with the following removal rates: TSS removal 75-90%, COD removal 60-70%, and BOD removal 60-75%. Optimum performance was achieved within minutes when units were started full (wet startup) but could be delayed for up to one hour when units were started empty (dry startup). The major concerns regarding the use of these processes were as follows:

- The Actiflo® system was seen to be more stable compared to the DensaDeg® process partly due to the reliance of the thickened sludge as a ballast compared to micro-sand.
- Treatment in the DensaDeg® system did not occur until a thick enough blanket of sludge had formed which created the necessary lag period before full treatment can be achieved. In contrast, the Actiflo® system reached optimal performance in minutes.
- Changes in rise rates often required adjustment in chemical feed rates for the DensaDeg® process. The Actiflo® system showed stable performance with variable rise rates, with little or no impact on chemical feed doses.
- The Actiflo® system requires a considerable amount of sand for operation. In large wet weather systems the inventory of sand could be in the order of hundreds of tons that will have to be worked on after every storm event. The disposal of this material was an issue that needs to be addressed.

- Subsequent to these piloting, additional pilot work was carried out in 2006 at the NPF based exclusively on the Actiflo® system (Jolis, 2006) to confirm its performance under real-time conditions during wet weather flows. The City is undertaking a 5 master planning study that will define the timelines for expansion of the NPF. However, based on discussions with SFPUC (Jolis, 2006), the Actiflo® process will be the system of choice.



Southeast Water Pollution Control Plant

North Point Facility

11.5.8 Windsor Retention Treatment Basin ESR and Functional Design/Pilot Plant

Stantec Consulting Ltd. was retained on behalf of the City of Windsor to undertake tasks related to the requirements defined in the Municipal Engineers Association document entitled "Municipal Class Environmental Assessment (June 2000)" for the City of Windsor Riverfront Retention Treatment Basins Evaluation. These tasks included review and analysis of existing reports and data, collection of additional data and information, discussions with various government agencies, municipalities, properly interested parties and the public regarding the problems with the existing CSO system, identifying and evaluating alternative solutions, functional design, reviewing and discussing these alternatives at various meetings with all concerned parties and selection of the recommended alternative.

Combined sewer overflows (CSOs) are a significant pollution source to the Detroit River. A pilot RTB, using polymer-aided flocculation for CSO treatment, was constructed and tested at the Lou Romano Water Reclamation Plant. Tests were conducted for eight CSO events between July and November 2001. Polymer and dosage effects on RTB effluent quality were investigated, and the link between overflow rate and total suspended solids removal for Windsor CSO was established. The results reveal that polymer significantly increases the surface-loading rate through the RTB, resulting in smaller treatment units.

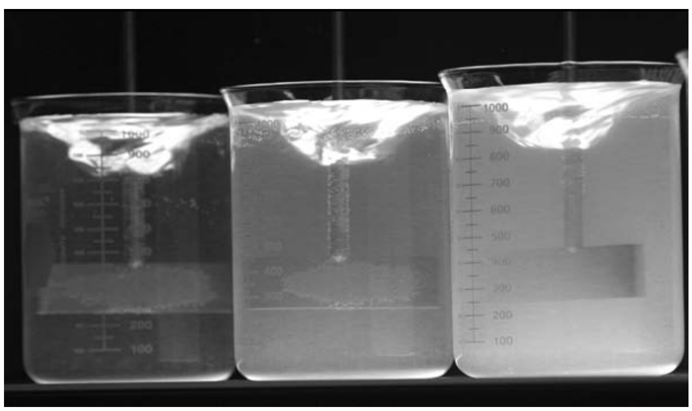
11.6 PRELIMINARY JAR TESTING

Due to lack of wet weather events in the summer of 2006, only one set of preliminary jar testing was possible at the SEWPCC. These tests were conducted on August 10, 2006 immediately following a rainstorm that resulted in flows in excess of 100 ML/d. At the time of sample collection, the flow was recorded as 104 ML/d at the SEWPCC and did not likely catch the first flush.

The raw wastewater sample was collected from the effluent of the grit removal facility. The jar tests were performed using Phipps & Bird apparatus equipped with six-paddle stirrer. As a first trial, three coagulants that could be sourced from a local supplier were selected for preliminary jar testing. This included: alum, ClearPac and ClearPac Plus in combination with an anionic polymer. Based on the review of other studies, preliminary chemical doses were selected as follows: 20 mg/L, 40 mg/L, 60mg/L, 80mg/L and 100 mg/L combined with 1.0 mg/L of polymer at all doses. A control sample was maintained with no dosage of either coagulant or polymer. Also no polymer dose was added to ClearPac Plus as it came premixed with a 10% solution of polymer. Each run of jar testing consisted of 1 minute of flash mixing at 100 rpm, 4 minutes of slow mixing at 20 rpm and 60 minutes of settling prior to decanting and sample testing.

The following parameters were tested:

- pH
- UV transmissivity
- Alkalinity
- Temperature
- BOD5
- TSS
- VSS
- TKN
- TP
- COD



The testing results are presented in Appendix F. The key results are also presented in Figures 11.11 to 11.15. The values on the y-axis show the residual contaminant concentrations in the decanted water sample.

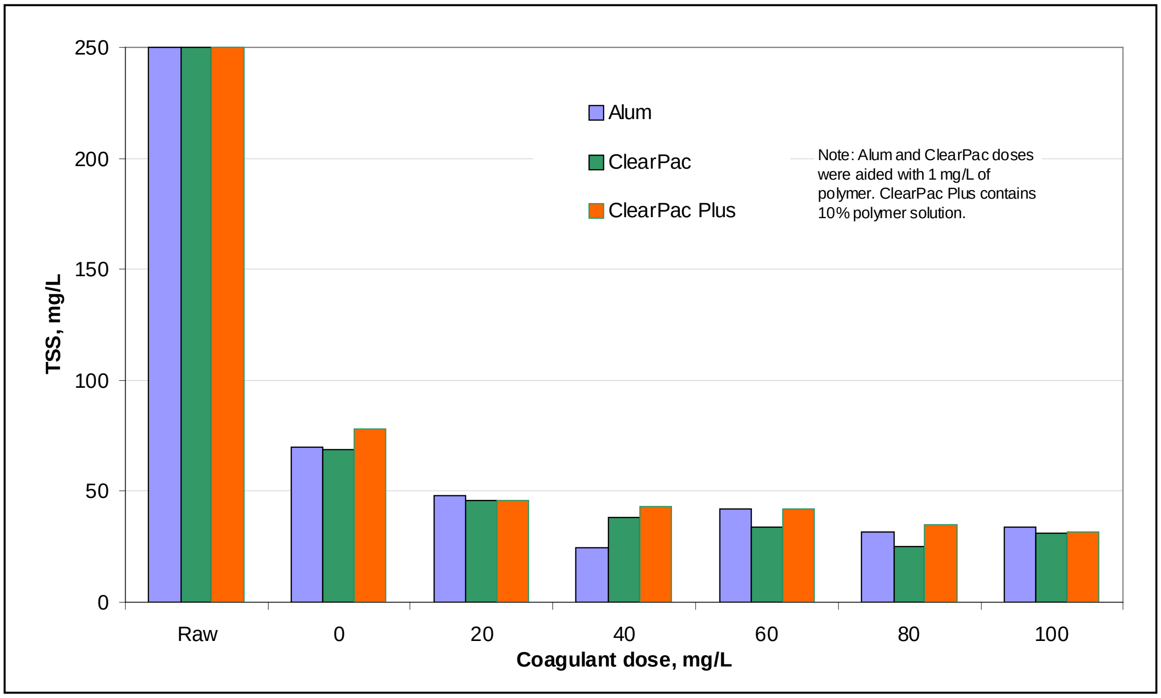


Figure 11.11 – TSS Removal

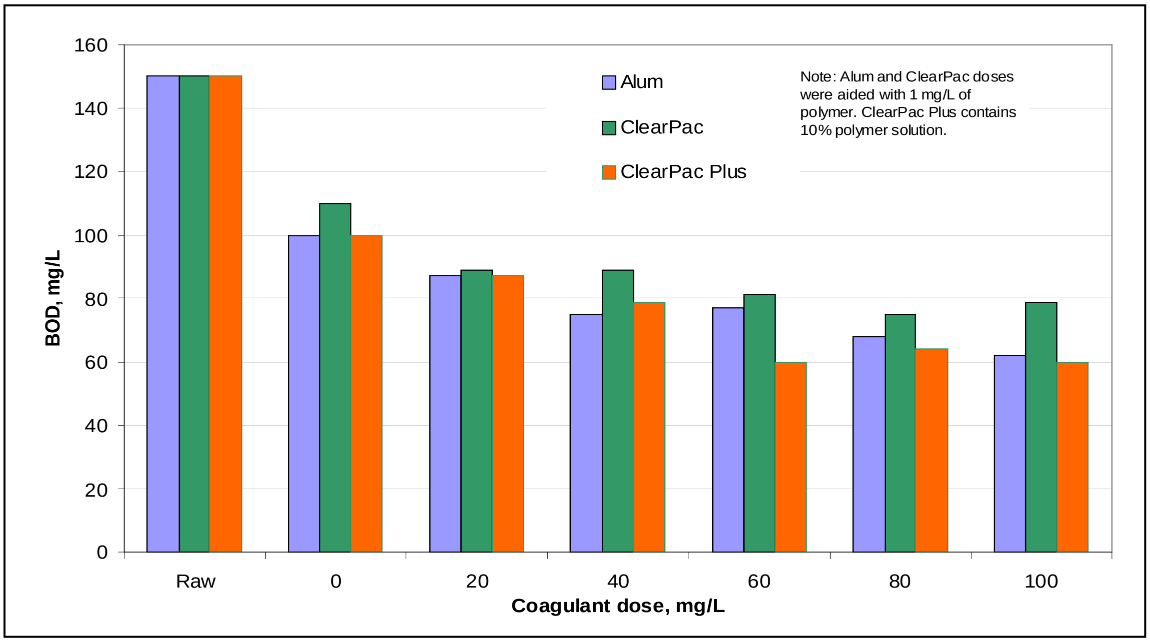


Figure 11.12 – BOD_5 Removal

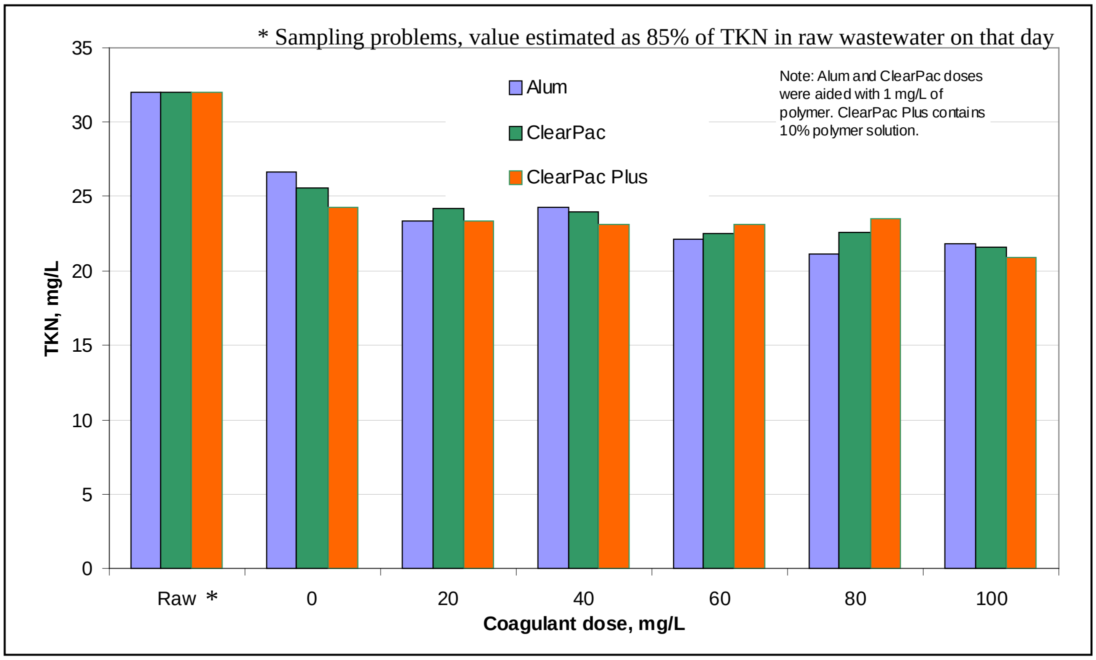


Figure 11.13 – TKN Removal

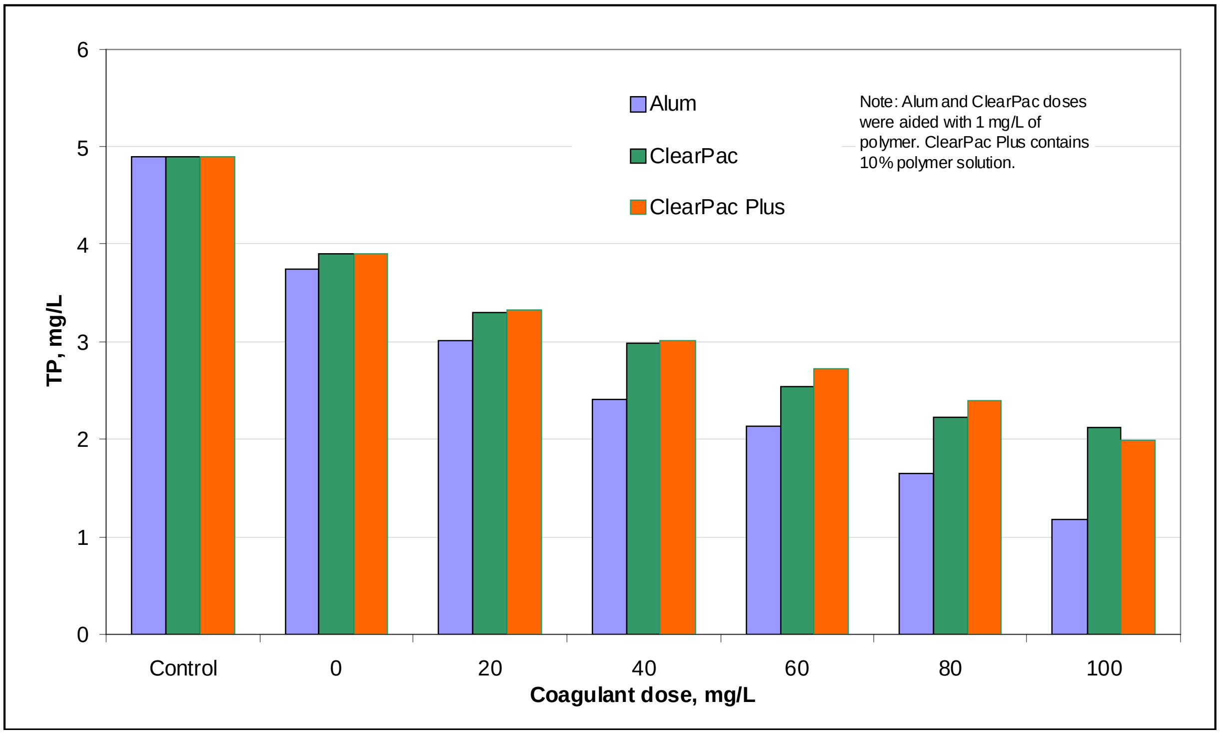


Figure 11.14 – Total Phosphorus Removal

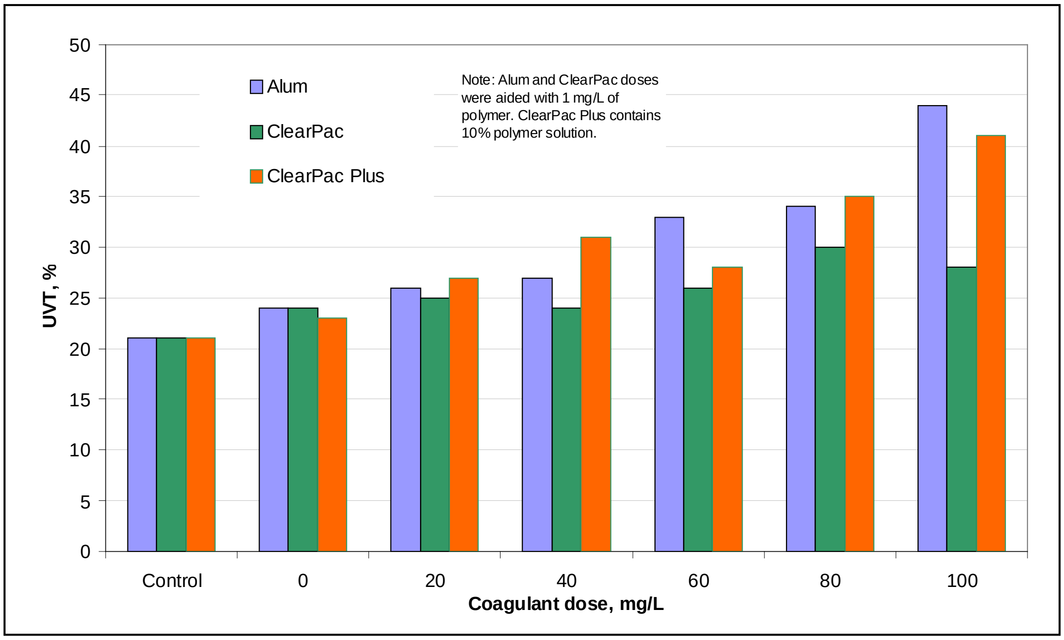


Figure 11.15 – Supernatant UVT (%)

11.6.1 Summary of Jar Test Results and Discussions

The following conclusions can be drawn from the single bench scale testing conducted for the SEWPCC raw wastewater under high flow conditions (> 100 ML/d):

- Preliminary jar testing indicates that the following efficiencies:
  - TSS = 69 to 90% (effluent TSS in the range of 78 ~ 24 mg/L)
  - BOD_5 = 27 to 60% (effluent BOD_5 in the range of 110 ~ 60 mg/L)
  - TP = 20 to 76% (effluent TP in the range of 3.9 ~ 1.2 mg/L)
  - TKN = 17 to 34 % (effluent TP in the range of 26.6 ~ 20.9 mg/L)
  - Maximum UV transmittance of 44% (decanted supernatant)
- Alum in combination with an anionic polymer provided the best performance compared to ClearPAC or ClearPAC plus.
- Although further optimization is necessary, alum dose in the range of 40 ~ 60 mg/L provided the best performance with respect to BOD, TSS.

- As expected, the percentage (%) of TP removal was directly proportional to the amount of chemicals used in the experiments. The highest TP removal was achieved with alum.
- The TKN removal was within reported range from previous studies. The data obtained from the jar tests show a 28.8% TKN removal on average and 34.7% maximum TKN removal. Historical data for the SEWPCC shows that ammonia (which is highly soluble) constitutes approximately 60% of TKN in raw sewage. The fraction of TKN that is soluble was approximately 74.5% based on 2006 data provided from the City.
- The % UVT of raw wastewater was 21%. On average, wastewater treated chemically achieved UVT of 30.6% with maximum UVT recorded at 44%. As expected, these values are significantly lower than the average UVT of 50.3% recorded for the secondary effluent at SEWPCC for the year 2006. For comparison purposes, UVT in the range of 50% is considered as poor for disinfection through UV irradiation, while UVT in the range of 70% is considered good. This preliminary bench scale results suggests that the wastewater following chemical pre-treatment is of very poor quality (with respect to UVT). Hence, UV disinfection of side-stream effluent may not be appropriate.

11.7 CONCLUSIONS

Based on discussions presented in this section, implementation of a side-stream process in the treatment of wet weather flows is considered appropriate for the SEWPCC facility.

Since the SEWPCC experiences very high wet weather flows relative to the average day flow, implementing a side-stream treatment process for these peak flows would protect the biological process from washout of the viable biomass, thereby maintaining optimal performance of the BNR process under such conditions. The quality of the treatment plant effluent can be restored immediately after the storm event. Secondly, the side-stream treatment systems cost about one-third the cost of a BNR system and the entire treatment facilities need not be oversized to handle these unusual flow events. This could result in a significant capital and operating cost savings for the City.

As discussed, clarification is often a key component to such wet weather treatment strategies. The choice of either a chemically enhanced primary clarification or high rate clarification process technologies is dictated by results of pilot testing and overall costs. Due to intermittent use of the side-stream process, the City may wish to select a technology that can also be used during dry weather flow if necessary. As an example, Lamella clarification can be used without chemicals as a part of routine operation of the primary clarification process. Actiflo® will have limited use during dry weather flows as the requirement of chemicals and disposal of sludge and microsand can make the process uneconomical. In addition, there are no space limitations at the SEWPCC that mandate a small footprint system such as Actiflo®.

On the other hand, Actiflo® has a better track record in North America with several operating facilities. Many of these installations are affected by high CSO type events and are subject to space limitations, making Actiflo® a right choice in that situation. Similarly, some facilities have located their high rate clarification process such as Actiflo® after the secondary treatment, where it can be used for tertiary TSS and TP removal during dry weather flows.

11.8 RECOMMENDATIONS

For the purpose of this section and facility layout, the Lamella plate clarification technology is recommended based on the following benefits:

- The EPT based on Lamella plate clarification technology can be used under both dry and wet weather operations for the SEWPCC. Given the limited number of wet weather events expected each year, and the capital expenditure involved for this option, the City may wish to make use of the Lamella plate clarifiers during dry weather conditions (no chemicals).
- Possible year round utilization of Lamella plate clarifiers maximizes the utilization of process resources.
- The overall primary effluent can be significantly improved due to lower operating SORs.
- The resulting primary sludge will have higher solids content because of the higher retention time in the clarifiers (both existing and proposed).
- The Lamella plate clarifiers could be operated as a back up to the existing primary clarifiers should maintenance is required.

11.9 FUTURE DIRECTIONS

Additional work is recommended to address the following issues:

- Detailed characterization of raw wastewater quality during wet weather events
- Bench scale testing to optimize chemical dosages and types
- Site visits to similar facilities such as Gold Bar and Longueil WWTP, Quebec to obtain feed-back on operational and maintenance issues associated with plate settler technology
- Pilot scale experiment of the Lamella plate clarification is strongly recommended. The pilot unit constructed by the City of Edmonton has a capacity 5 ML/d and is available for SEWPCC. The photo of the pilot plant is shown below. Pilot plat operations is necessary to address the following:
  - Optimal level of treatment with respect to BOD_5, TSS, TP and TKN removal
  - Chemical optimization
  - Sludge production

- Potential for plugging of lamella plates and the performance of cleaning mechanisms
- Grease and scum removal

Figure 11.16 - Gold Bar WWTP - Plate Settler Pilot Plant (Source: Stantec)

				Treatment, mg/L					
Parameter	*Units*	Control	Raw data plant August 10	Alum 0/Polymer 0	Alum 20/Polymer 1	Alum 40/Polymer 1	Alum 60/Poymer 1	Alum 80/Polymer 1	Alum 100/Polymer 1
Alkalinity	*mg/L*	208	240	212	205	193	184	176	167
Dissolved Ammonia	*mg/L*	16.4		16.8	17.0	17.0	17.0	17.0	17.1
TKN	*mg/L*	17.9	32	26.6	23.4	24.3	22.1	21.1	21.8
BOD	*mg/L*	150	148	100	87	75	77	68	62
COD	*mg/L*	210	248	240	190	180	170	120	110
TP	*mg/L*	4.89	5.27	3.75	3.02	2.41	2.14	1.65	1.17
TSS	*mg/L*	250		70	48	24.3	42	32	34
VSS	*mg/L*	58		51	34	31	30	21	22
pH		7.7		7.6	7.6	7.5	7.4	7.4	7.3
UV Transmittance	%	21		24	26	27	33	34	44
ClearPac + Polymer				Treatment, mg/L					
Parameter	*Units*	Control		ClearPac 0/Polymer 0	ClearPac 20/Polymer 1	ClearPac 40/Polymer 1	ClearPac 60/Poymer 1	ClearPac 80/Polymer 1	ClearPac 100/Polymer 1
Alkalinity	*mg/L*	208	240	213	207	204	203	199	198
Dissolved Ammonia	*mg/L*	16.4		17.0	17.1	17.0	16.9	16.8	17.1
TKN	*mg/L*	17.9	32	25.6	24.2	23.9	22.5	22.6	21.6
BOD	*mg/L*	150	148	110	89	89	81	75	79
COD	*mg/L*	210	248	260	160	170	160	160	160
TP	*mg/L*	4.89	5.27	3.91	3.3	2.98	2.54	2.23	2.12
TSS	*mg/L*	250		69	46	38	34	25	31
VSS	*mg/L*	58		39	25	32	30	23	27
pH		7.7		7.7	7.6	7.7	7.6	7.6	7.6
UV Transmittance	%	21		24	25	24	26	30	28
ClearPac Plus				Treatment, mg/L					
Parameter	*Units*	Control		ClearPac Plus 0	ClearPac Plus 20	ClearPac Plus 40	ClearPac Plus 60	ClearPac Plus 80	ClearPac Plus 100
Alkalinity	*mg/L*	208	240	211	207	205	202	200	197
Dissolved Ammonia	*mg/L*	16.4		17.2	17.1	17.0	16.8	16.8	16.8
TKN	*mg/L*	17.9	32	24.3	23.4	23.1	23.1	23.5	20.9
BOD	*mg/L*	150	148	100	87	79	60	64	60
COD	*mg/L*	210	248	200	190	160	150	130	110
TP	*mg/L*	4.89	5.27	3.91	3.32	3.01	2.72	2.4	1.99
TSS	*mg/L*	250		78	46	43	42	35	32
VSS	*mg/L*	58		51	38	38	35	29	30
pH		7.7		7.7	7.7	7.7	7.6	7.7	7.7
UV Transmittance	%	21		23	27	31	28	35	41

APPENDIX E

Biosolids Calculations

Table A – HRC

Process Step	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
HIGH RATE CLARIFICATION	11.2	1,730	1,602	81.2%	1405	324	47
Description:	Prel. Clarification with ferric chloride + polymer feed.						
Effect upon flow:	No effect on flow. Backwashing produces waste stream that is thickened and treated w/ plant sludge.						
Assume:	Coag aids increase TSS removal efficency to 85%						
Polymer Feed =	1	mg/L *	Flow =	11.2	ML/d		
Ferric Chloride feed =	75	mg/L *	Flow =	840	kg/d		
Sludge conc. =	10	g SS/L					
Removal Efficiencies:		TSS Removal	BOD_5 Removal			TKN Removal	TP Removal
Assume:		85.0%	60.0%			40%	80%
Chemical Sludge Mass =	50% $FeCl_3$ feed =		420	kg/d			
Total Sludge Mass =	TSS Loading + Chem Sludge =			2150	kg/d		
Next Process Step(s):	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	NH_4 Loading (Kg/d)	TP Loading (Kg/d)
Assume:	Organic N converted to ammonia.						
Effluent to BAF:	11.2	323	641	65.4%	210	194	9
HRC Sludge to Blend Tank:	0.186	1,828	961	65.4%	1,195	129	38
Flow(Primary Sludge) =	TSS Load * s.g. / conc.						

Table B – CEPT

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
PRIMARY SETTLER FEED	160	24,796	22,761	80%	19837	4,631	661
Description:	Horizontal Basins (3 existing and 1 new)						
	Chemically Enhanced using Ferric Chloride						
Effect upon flow:	Settled solids reduces SS and BOD_5, no impact on flow.						
Calculations:	Settled solids content of sludge at 4%.						
Ferric Chloride solids =	40	mg/L *	Flow =	6,400	kg/d		
Assume:	35%	increase in sludge mass from $FeCl_3$ added [Ref. WEF MOP-8]					
SS + Chem. Sludge Mass =	1.35* (TSS Load) =		33475	kg/d			
Prim. Sludge Flow =	TSS * s.g. / sludge concentration						
Removal Efficiencies:		TSS Removal	BOD_5 Removal			TKN Removal	TP Removal
		50.0%	35.0%			12%	48%
Next Process Step(s):	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Next Process Step(s):							
Prim. Effluent pumped to BAF:	160	16,737	14,795	45%	7,535	4,071	342
Prim. Sludge to Tanks:	0.41	16,737	7,966	73.5%	12,302	560	319

Table C – Intermediate Pumping Station

Process Step	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
INTERMEDIATE PUMPING							
Main Flow Primary Effluent:	150	15,640	13,946	64.50%	10,072	3,683	158.9
Clarified Backwash Return:	17.2	4,198	3,558	86.90%	3,648	1,814	7.0
HRC Effluent:	11.2	322.5	640.8	65.40%	209.8	194.1	9.5

Table D – NDN BAF

Process Step	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BIOFILTER NDN	178.0	20,160	18,145	69%	13,929	5,692	175
Design Basis:	**10 cells of 180 m^2**						
Description:	Aerated biological filter with recycle of nitrified eff.						
	Daily BW Vol. of Stage 1 NDN filters (10 cells @ 1575 m^3 water for backwash/filter						
Effect upon flow:	100% of flow is treated						
Calculations:	(Expected Performance based on PSR)						
TSS, out =	15	Mg/L*Q =	2,670	kg/d			
BOD_5, out =	20	Mg/L*Q =	3,560	kg/d			
NH_4, out =	4	Mg/L*Q =	712	kg/d	TKN		
TP, out =	0.9	Mg/L*Q =	160	kg/d			
Calculations:	Solids production est'd at 0.4 g /g BOD_5 + 0.65 g /g TSS [Ref EPA Tech Assessment of the BAF, 90]						
Assume:	BOD_5 removed during BW of BAF = 60% of Total BAF feed By-Pass Flow = 50% of BAF (effluent)						
Next Process Step(s):	**Flow (MLD)**	**TSS Loading (Kg/d)**	**BOD_5 Loading (Kg/d)**	**VSS (as % of TSS)**	**VSS Loading (Kg/d)**	**NH_4 Loading (Kg/d)**	**TP Loading (Kg/d)**
Biofilter NDN Backwash:	15.75	20,362	10,887	88%	17,919	2,862	29
Biofilter Post DN By-Pass (50%):	81.1	1,217	1,623	88%	1,071	453	73.0
Biofilters – Post DN:	81.1	1,217	1,623	88%	1,071	453	73.0

Table E – Post DN BAF

Process Step	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BIOFILTER POST NDN	81.1	1,217	1,623	88%	1,071	453	73
Design Basis:	2 cells of 87 m^2 each						
Description:	Aerated Biological Filter with methanol feed for denitrification						
	Daily BW Vol. of Stage 2 filters (2 @ 525 m^3 filter)						
Effect upon flow:	50% of flow is treated in the Post DN process.						
Calculations:							
TSS, out =	15	Mg/L*Q =	1,217	kg/d			
BOD_5, out =	15	Mg/L*Q =	1,217	kg/d			
NH_4, out =	4	Mg/L*Q =	325	kg/d			
TP, out =	0.8	Mg/L*Q =	65	kg/d			
Methanol Feed =	2.45	Mg/L*Q =	199	kg/d			
Next Process Step(s):	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	NH_4 Loading (Kg/d)	TP Loading (Kg/d)
Biofilter Post DN Backwash:	1,050	1,520	974	88%	1,337	160	8
UV Disinfection	80.1	1,217	1,217	88%	1,071	453	65

Table F – NDN BAF - Backwash Process Stream

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BIOFILTER NDN BACKWASH	17.850	21,882	11,860	88%	19,256	3,023	37
Description:	Daily BW Vol. of Stage 1 filters (10 @ 1575 m^3/filter)						
Effect upon flow:	22,050 m^3/d est'd backwash water requirement.						
Calculations:	Solids production est'd at 0.4 g /g BOD_5 + 0.65 g /g TSS. [Ref EPA Tech Assessment of the BAF, 90]						
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Backwash Waste Storage	17.850	21,882	11,860	88%	19,256	3,023	37

Table G –Post DN BAF - Backwash Process Stream

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BIOFILTER POST DN BACKWASH	2.100	1,520	974	88%	1,337	160	8
Description:	Daily BW Vol. of Stage 2 filters (3 @ 525 m^3/filter)						
Effect upon flow:	1,575 m^3/d est'd backwash water requirement.						
Calculations:	Solids production est'd at 0.4 g /g BOD_5 + 0.65 g /g TSS. [Ref EPA Tech Assessment of the BAF, 90]			Note: Methanol is included in BOD_5 calcs			
Assume:	BOD_5 Removed during BW of BAF = 60% of Total BAF Feed						
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Backwash Storage Tank	2.100	1,520	974	88%	1,337	160	8

Table H – Backwash Clarification

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BACKWASH CLARIFIERS	17.85	21,882	11,860	88%	19,256	3,023	37
Description:	Prel. Clarification with ferric chloride + polymer feed.						
Effect upon flow:	No effect on flow. Backwashing produces waste stream that is thickened and treated w/ plant sludge.						
Calculations:	Typ. removal efficiency w/o chemical enhancement:						
	50% TSS Load; 30% BOD_5 Load						
Assume:	Coag aids increase TSS removal efficiency to 85%						
Polymer Feed =	1	mg/L *	Flow =	24	kg/d		
Ferric Chloride feed =	25	mg/L *	Flow =	604	kg/d		
Sludge conc. =	25	g SS/L					
Removal Efficiencies:		TSS Removal	BOD_5 Removal			TKN Removal	TP Removal
Assume:		80.0%	70.0%			40%	80%
Chemical Sludge Mass =	50% $FeCl_3$ feed =		302	kg/d			
Total Sludge Mass =	TSS Loading + Chem Sludge =			28053	kg/d		
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Sludge Blend Tank:	0.69	17,684	8,302	87.11%	15,405	1,209	30
Clarifier Effluent to Int. Pumps:	17.16	4,198	3,558	86.89%	3,648	1,814	7

Table I – Sludge Blend Tank

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
NEW SOLIDS BLENDING TANK							
Prim. Settled Sludge	0.383	15,640	7,509	65%	10,088	650	477
High Rate Clarifier Solids	0.186	1,828	961	65%	1,195	129	38
Settled Backwash Solids	0.69	17,684	8,302	87%	15,405	1,209.1	30
Description:	Mixing Tank of various solids stream flows.						
Effect upon flow:	Mixing has no impact upon solids characteristics.						
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Blended Sludge to Thickening:	1.263	35,151	16,773	76%	26687	1,989	545

Table J – Sludge Thickening

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
BLENDED SOLIDS THICKENING	1.26	35,151	16,773	76%	26,687	1,989	545
Description:	Sludge thickening						
Blended Solids concentration	2.73	%					
where:		Prim sludge conc =	4.0%				
and		Settled BW solids conc =	2.5%				
Assumptions:		Anaerobic Digestion % Solids Concentration Desired:	5.00				
		Specific Gravity of TSS:	1.02				
Solids Capture Efficiency:		95%					
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Anaerobic Digestion:	0.65	33,393.49	15,934	76%	25,353	1,889	517
Filtrate from Thickening to Centrate Treatment:	0.61	1,758	839	76%	1,334	99	27

Table K – Anaerobic Digestion

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
ANAEROBIC DIGESTER (2 stage)	0.65	33,393	15,934	76%	25,353	1,889	517
Description:	Two stage mesophilic anaerobic digester						
Assumptions:							
Specific Gravity of TSS	1.03						
Solids retention time	20	days					
Volatile Solids Loading	2.2	kg VS / m^3*d					
Safety Factor: f.s. .(volume)	1.1	(this is based upon grit and foaming)					
VS Destruction	45.00	%					
Effect upon flow:	Reduction of volatile solids, no changes to hydraulic conditions						
Calculations:							
VS Reduction:	$\%VS_d = (1 - Vs_{out}/Vs_{in})*100\%$			[Ref: Van Kleck]			
Digester Volume by VS load basis:			**VSS Load/ VS Loading Rate* f.s =**			**12.7 ML**	
Digester Volume by SRT:	No recycle of solids therefore SRT and HRT the same						
Digester Volume by SRT or HRT basis:			**Flow * Solids Ret. Time * f.s =**			**14.4 ML**	
Design Digester Volume is max of the above two approaches						**14.4 ML**	
Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
Sludge Dewatering	0.65	21,985	4,525	63%	13,944	1,889	517

Table L – Sludge Dewatering

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
CENTRIFUGE DEWATERING	0.655	21,985	4,525	63%	13,944	1,889	517
Description:	Centrifuge Dewatering						
Effect upon flow:	Dewatering systems are anticipated to include centrifuges. Standard polymer feed systems that utilize a cationic polymer. Feed solids are of standard characterization of a mix of primary and waste activated secondary solids. Capture of 95% of solids concentration is anticipated.						
Calculations:	S.C. = TSS Load * s.g. /Sludge Flow =					3.46%	
Assumptions:							
Specific Gravity of TSS:			1.03				
Post Dewatering Solids Concentration (% solids):			25				
Solids Capture Efficiency:			95%				
Work Week for Dewatering:			7	days			
Period of Operation for Dewatering			8	hours/ day			
Weekly Volume of Digested Solids			4.58	ML/ wk			
Flow rate to Dewatering Equipment			115	m3/hr			
Solids Loading Rate:			3847	Kg/hr			

Note: Numbers below are based on 7 days dewatering

Next Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)
To Centrate Treatment (Centrate):	0.6	1,099	226	1	697	459	30
Trucked Offsite (solids):	0.08	20,885	4,299	1	13,247	1,430	487

Table M – Centrate Treatment

Process Step(s)	Flow (MLD)	TSS Loading (Kg/d)	BOD_5 Loading (Kg/d)	VSS (as % of TSS)	VSS Loading (Kg/d)	TKN Loading (Kg/d)	TP Loading (Kg/d)	TEMP
Thickener Filtrate	0.61	1,758	839	76%	1,334	99	27	15
Centrate	0.57	1,099	226	63%	697	459	30	38
TOTAL	1.2	2,857	1,065	69.86%	2,032	558	58	26

Table N – Mass Balance Summary Table

		Spring Loading Conditions (Max. Month Average)							
	Flow / Solids	Flow Average (MLD)	TSS (Kg/day)	BOD_5 (Kg/day)	VSS (as %SS)	VSS (Kg/day)	Solids Conc. g SS/L	Specific Gravity	**Solids (dry MT/ day)**
A.	Raw Influent WW	160.0	24,796.0	22,761.0					**24.8**
B.	Pretreated (PT) WW	160.0	24796	22761					**24.8**
C.	Primary Clarifer Feed - Main	160.0	24796	22761					**24.8**
a.	Ferric Chloride feed		6400				0.04		**6.4**
D.	Primary Clarifier Effluent	160.0	15640	13946					**15.6**
E.	Primary Clarifier Sludge	0.38	15640	7509	64.5%	10088	40	1.02	**15.6**
F.	NDN Biofilter Feed	178.0	20160	18145					**20.2**
G.	DN Biofilter Feed	81.1	1217	1623					**1.2**
b.	Methanol Feed		199				0.00245		**0.2**
H.	DN Biofilter By-Pass	81.1	1217	1623					**1.2**
I.	Backwash 1, NDN Biofilter	15.75	20362		88%	17919			**20.4**
J.	Backwash 2, DN Biofilter	2.10	1520		88%	1337			**1.5**
K.	Backwash Total	17.85	21882			19256			**21.9**
c.	Ferric Chloride feed		446				0.025		**0.4**
d.	Polymer feed		17.85				0.001		
L.	Settled Solids from BW	0.69	17684		87%	15405	25	1.02	**17.7**
M.	Backwash Return Flow	17.16	4198	3558		3648			**4.2**
N.	Overflow to Ballasted Floc Clar	-	0	0		0			**0.0**
e.	Ferric Chloride feed		0				0.075		**0.0**
f.	Polymer feed		0				0.001		**0.0**
O.	Settled Sludge - Ballasted Floc	0.19	1828	961	65%	1195	10	1.02	**1.8**
P.	Total Sludge Feed to Thickener	1.26	35,151		76%	26,687			**35.2**
Q.	Thickener Return Flow	0.61	0	0		0	-	1.00	**0.0**
R.	Digester Feed Total	0.65	33393		76%	25353			**33.4**
S.	Digested Biosolids	0.65	21985		63%	13944			**22.0**
T.	Dewatered Biosolids	0.08	20885		63%	13247			**20.9**
U.	Pre-AD Thickener Filtrate	0.61	1758	839		1334			**1.8**
V.	Centrifuge Centrate	0.57	1099	226		697			**1.1**
W.	Total Dewatering Return Flow	1.79	2857	1065	71%	2032			**2.9**

APPENDIX F

Major Equipment
Procurement Packages

APPENDIX F

Major Equipment Procurement Technical Specifications for Biologically Active Filtration (BAF) and High Rate Clarification (HRC).

The documents for the supply of these major equipment is found under separate cover, the cover pages are provided for reference purposes only.

THE CITY OF WINNIPEG

REQUEST FOR PROPOSAL

RFP NO. ^

REQUEST FOR PROPOSAL FOR THE SUPPLY OF BIOLOGICALLY ACTIVE FILTRATION EQUIPMENT FOR THE SOUTH END WATER POLLUTION CONTROL CENTRE EXPANSION / UPGRADING PROJECT

Winnipeg

THE CITY OF WINNIPEG

REQUEST FOR PROPOSAL

RFP NO. ^

REQUEST FOR PROPOSAL FOR THE SUPPLY OF HIGH RATE CLARIFICATION EQUIPMENT FOR THE SOUTH END WATER POLLUTION CONTROL CENTRE EXPANSION / UPGRADING PROJECT

APPENDIX G

Site Survey