CORNWALL WASTEWATER TREATMENT PLANT
ENVIRONMENTAL ASSESSMENT UPDATE

TECHNICAL MEMORANDUM NO. 5
LIQUID TREATMENT TRAIN EVALUATION

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1.0 INTRODUCTION

The Cornwall Wastewater Treatment Plant (WWTP) is owned and operated by the City of Cornwall. The plant provides primary treatment with chemical addition for enhanced treatment and phosphorus removal as well as disinfection. The existing WWTP has a Certificate of Approval (C of A) average day flow (ADF) capacity of 54,432 m^3/d and a peak design capacity of 108,864 m^3/d. Biosolids are treated on site in an anaerobic digestion process and dewatered using centrifuges, prior to disposal in the City's landfill.

The Cornwall WWTP was originally constructed in 1968 and has undergone several expansions since then. The WWTP serves approximately 46,000 people, services, commercial and industrial properties and receives leachate from five waste disposal sites. Plant ownership was transferred to the MOE in 1970 and then to the City of Cornwall in 2000. Plant expansion in 1988 resulted in the current plant layout.

Figure 1 presents an aerial image of the Cornwall WWTP.

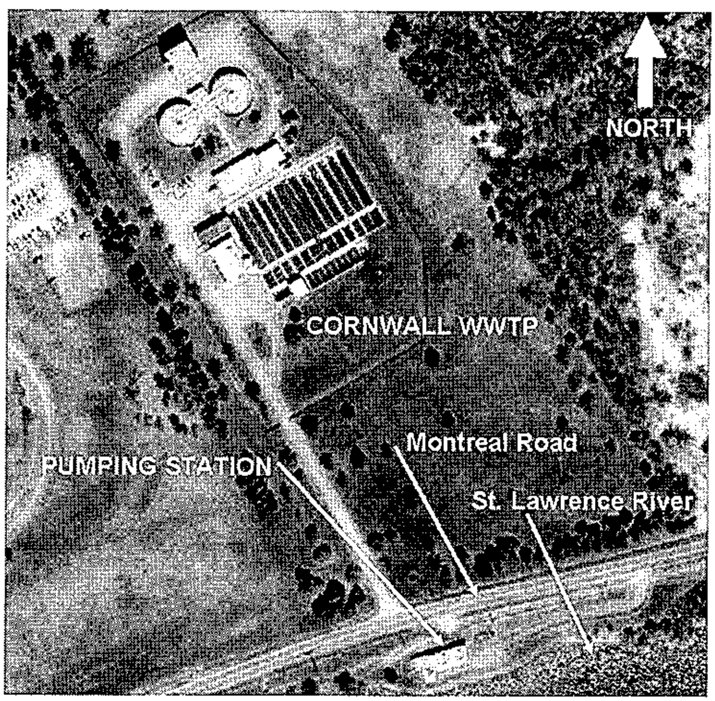

Figure 1 - Satellite Photograph of the Cornwall WWTP (courtesy of GoogleMaps)

A Pollution Control Planning (PCP) study was undertaken in 1995 to address concerns raised as part of the Remedial Action Plan (RAP), which stemmed from the designation of the St. Lawrence River (Cornwall area) as an Area of Concern (AOC) by the Water Quality Board and the International Joint Commission. The PCP is currently being updated to reflect current conditions including a hydraulic assessment of critical sewer infrastructure and to determine compliance with MOE Procedure F-5-5. In addition to other recommendations, the RAP recommended the following upgrades to the Cornwall WWTP:

- Upgrading the existing primary treatment plant to the equivalent of secondary treatment
- Achieve Total Phosphorus removal to a compliance of 1.0 mg/L, with an objective of 0.5 mg/L
- Increase removal efficiency of other toxic contaminants
- Reduce bacteria levels

Following a plant-wide evaluation for required upgrades in 2003, a Schedule C Class Environmental Assessment (Class EA) for these upgrades was completed in 2005. As a result of the Class EA, a Biological Aerated Filter (BAF) system was recommended for the secondary treatment process upgrade and Ultraviolet (UV) irradiation was recommended for the disinfection system.

In May 2009, the City retained J.L. Richards & Associates Limited, in association with XCG Consultants Ltd. and CH2M HILL Canada Limited, to update the 2005 Class EA and to update the budget for capital-required upgrades as well as life-cycle costing over the 20 year EA planning period. The update will revisit the preferred treatment technologies identified in the EA based on existing conditions and constraints and provide updated construction and life-cycle cost estimates.

The purpose of Technical Memorandum No. 5 (TM5) is to examine and evaluate the existing treatment processes as well as to make recommendations for the most suitable technology for the upgraded plant. These technologies will be compared and evaluated to determine a recommended alternative for the upgraded plant. The findings of the previous EA will be taken into account, as well as the updated information gathered and summarized in TM2. In addition to the primary and secondary treatment process evaluation, a comparison between ultraviolet irradiation (UV) and chlorination/dechlorination technologies as they would be applied to the

Cornwall WWTP will be made. This technical memorandum will form part of the Environmental Study Report Addenda (ESR Addenda), which is anticipated to be the main deliverable for this project.

2.0 CONCEPTUAL LEVEL DESIGN BASIS

2.1 Design Flows

The design average day flow (ADF) flow is based on the existing Certificate of Approval (C of A) rated ADF capacity of 54,432 m^3/d and provisions to allow for 20 percent growth in Cornwall.

As discussed in Technical Memorandum No. 3, it is possible to increase the pumping station capacity to approximately 186,000 m^3/day while maintaining a forcemain velocity of 3.0 m/s or less. Following discussions with City staff, it was recommended that consideration be given to increasing the peak flow for the upgraded plant up to 160,000 m^3/d. This will be reviewed further during the design phase based on value engineering and a comprehensive cost-benefit analysis. In any event, the peak flow design basis shall be 130,000 m^3/d, as a minimum, consistent with the 2005 ESR. It should be noted that the collection system is such that peak flows can be sustained for extended periods during wet weather events. Given that all flows to the plant are pumped, the recommended peak day and peak instantaneous flow capacity are the same.

2.2 Design Concentrations

Based on the required provisions for 20 percent growth, and without further identification of growth specific to industry and population growth in Cornwall, the design raw sewage concentrations were based on historic concentrations and application of historic maximum month loading factors. Adopting these values as the design basis is intended to provide some safety factor and ensure that the design flows would include maximum month loadings.

The Cornwall WWTP currently receives landfill leachate from five landfill sites (one active site), and industrial wastewater from two major industries. The historic (and design basis) concentrations include the contributions from landfill leachate and industrial wastewater. It should be noted that any significant future variation in landfill leachate and/or industrial wastewater contributions could impact the raw sewage strength.

2.3 Summary of Conceptual Level Design Basis

A summary of the conceptual level raw sewage design basis for the Cornwall WWTP is presented in **Table 1**.

Table 1 – Cornwall WWTP Design Basis – Raw Sewage Characteristics

Parameter	Updated Design Basis	2005 ESR
Average Day Flow	65,318 m^3/d	65,318 m^3/d
Peak Flow[1]	up to 160,000 m^3/d [2]	130,000 m^3/d
Peak Flow Factor[1]	2.45	2.0
BOD_5 Concentration	110 mg/L	180 mg/L
TSS Concentration	165 mg/L	200 mg/L
TP Concentration	4 mg/L	5.0 mg/L
TKN Concentration	26 mg/L	n/a

Notes:
n/a – not available
1. Peak Day and Peak Instantaneous
2. Peak Day and Peak Instantaneous shall be 130,000 m^3/d, as a minimum, consistent with the 2005 ESR.

For secondary treatment design options that receive primary clarifier effluent as their influent stream, a conservative percentage reduction of 35% and 65% is applied to BOD_5 and TSS respectively.

2.4 Future Effluent Limits

The proposed effluent limits determined in Technical Memorandum 1B: Assimilative Capacity Assessment are presented in **Table 2**. The background and rationale for the proposed effluent limits are discussed in detail in Technical Memorandum 1B.

It should be noted that some removal of ammonia nitrogen would be required year round to meet the effluent ammonia requirements.

The treatment objectives were used to evaluate the performance of the reviewed secondary treatment technologies.

Table 2 – Cornwall WWTP Proposed Future Effluent Limits

Parameter	Proposed Future Effluent Limits	
	Objective	Compliance
$cBOD_5$ Concentration	15.0 mg/L	25.0 mg/L
TSS Concentration	15.0 mg/L	25.0 mg/L
TP Concentration	0.5 mg/L	0.8 mg/L
TKN Concentration January – March April – September October - December	 7.0 mg/L 5.0 mg/L 9.0 mg/L	 9.0 mg/L 7.0 mg/L 11.0 mg/L
E. coli.	100 CFU/100 mL	200 CFU/100 mL
Total Residual Chlorine	0.02 mg/L	0.04 mg/L

2.5 Existing Treatment Plant

The existing Cornwall WWTP consists of mechanically-cleaned fine screens upstream of raw sewage pumping. Raw sewage then passes through grit removal in the grit tanks prior to primary treatment in four rectangular primary clarifiers. Chemical coagulants are added upstream of the primary clarifiers for phosphorus removal and to enhance solids removal. Primary clarified effluent is disinfected by chlorination prior to discharge to the St. Lawrence River. Primary sludge is stabilized in anaerobic digesters, dewatered in two centrifuges and hauled to landfill for disposal.

2.5.1 Primary Treatment Capacity

There are four rectangular primary clarifiers at the Cornwall WWTP. Two primary clarifiers have a combined surface area of 1,361 m^2 and the remaining two have a combined surface area of 1,463 m^2, for a total primary clarifier surface area of 2,824 m^2.

The importance of thickening waste activated sludge produced by an activated sludge treatment process, prior to further processing, is well recognized. Thickening can reduce the volume of the WAS stream, which is typically 0.5 to 1.5 percent total solids (Metcalf and Eddy, 2003), to 50 to 25 percent of the original volume. This volume reduction minimizes the capital and operating cost for the downstream sludge processing, for example digester volume or dewatering equipment capacity, as well as operating, chemical and energy requirements.

The capital and operating costs of separate mechanical WAS thickening can be substantial. The commonly used methods for WAS thickening processes, such as dissolved air floatation (DAF), rotary drum thickeners, gravity belt thickeners or centrifuges, require significant energy input and chemical addition to improve solid liquid separation. Separate gravity thickening, although more cost effective from the operating standpoint than the other separate WAS thickening processes, is not well suited for the thickening of WAS, which has relatively poor settling and compaction characteristics when compared to the raw and mixed sludges.

The practice of thickening waste activated sludge in the primary clarifiers, which is termed co-thickening, is a cost effective thickening approach that could be employed in place of separate WAS thickening at facilities that have sufficient primary clarification capacity. WAS solids settle at a significantly slower rate than raw sewage solids. Therefore, hydraulic loading limits on primary clarifiers that are used for co-thickening WAS are substantially lower than those on primary clarifiers that do not co-thicken.

Co-thickening is particularly attractive to smaller wastewater treatment plants because they can avoid the high capital and operating cost of a separate WAS thickening process. Treatment facilities that do not have adequate primary clarification capacity, or were originally installed without primary clarifiers, for example SBR's or other extended aeration processes, sometimes employ separate thickening of WAS. However, the majority of these types of plants direct the WAS to aerobic digesters at low concentrations (0.5 to 1.0% TS) and then gravity-thicken the aerobically digested sludge in the sludge storage tank or secondary digester.

Separate WAS thickening is more cost effective for larger facilities. It can offer the advantage of producing a higher concentration of WAS to feed the downstream sludge stabilization or dewatering processes, eliminate the recycling of BOD_5 or suspended solids from the primary clarifier into the secondary process, and can minimize the re-inoculation of the secondary plant with filamentous organisms. Ontario wastewater treatment plants that have been designed with separate WAS thickening facilities are generally very large plants (Toronto's Ashbridges Bay and Highland Creek TPs, Peel's Lakeview WWTP, the City of Hamilton's Woodward Avenue STP, the City of Ottawa's ROPEC, and the City of London's Greenway WWTP) or facilities that when expanded did not have adequate primary clarifier capacity (Petawawa STP, the upgraded Kingston West WPCP, the Fort Erie Anger Avenue WPCP and the Collingwood WPCP). The vast majority of conventional activated sludge plants in Ontario co-thicken WAS and have done so successfully for many years.

Co-thickening in primary clarifiers will produce sludge with total solids (TS) concentration in the range of 3 to 4 percent, and in some cases up to 6 percent. This is approximately 1 to 2 percent lower TS concentration, when compared to raw primary sludge without the added WAS. When co-thickening, there is an increased risk of solids carry-over and higher TSS and BOD loading to the biological treatment process, especially when operating at high hydraulic loading rates. These impacts, however, are transient and can be alleviated by wasting sludge at low flow rates, co-thickening during low flow periods, and prompt sludge withdrawal to minimize solubilization of BOD_5 and re-suspension and carry-over of sludge solids. A properly operated co-thickening process should have a minimal impact on the liquid treatment process.

The advantages and disadvantages of co-thickening are summarized in **Table 3**.

Table 3 – Advantages and Disadvantages of WAS Co-Thickening

Advantages	Disadvantages
• reduced solids treatment process complexity • significant capital and operating cost savings compared to mechanical or gravity thickening technologies • no additional land requirements • no requirement to store WAS prior to thickening or to store thickened WAS prior to digestion, eliminating a potential odour source • no additional operating or maintenance labour requirements	• reduced raw sludge TS concentration by approximately 1%-2% TS, when compared to the concentration of raw primary sludge without WAS • potential for increased solids carry-over and TSS and BOD loading to secondary treatment (aeration) at high co-thickening rates • potential for recycling undesirable filamentous organisms to the secondary plant • reduced primary clarifier design surface overflow rate at peak flow from 60-80 m^3/m^2d to 50-60 m^3/m^2d (MOE, 2008)

The capacity of the existing primary treatment process was reviewed based on the process design data and the 2008 MOE Design Guidelines for primary sedimentation tanks. **Table 4** presents the treatment capacity of the existing primary sedimentation tanks operating with and without WAS co-thickening.

If the Cornwall WWTP were to incorporate co-thickening of waste activated sludge (WAS) in the primary sedimentation tanks, the peak daily flow capacity of the primary clarifiers would be 169,500 m^3/d. This value is greater than the design peak flow capacity of 160,000 m^3/d. Therefore, the existing primary clarifiers have sufficient capacity to co-thicken WAS and treat the future flows to the Cornwall WWTP.

Table 4 – Cornwall WWTP Primary Treatment Capacity

Design Parameter	MOE Guideline Value	Equivalent Peak Daily Flow Capacity [(1)]
Without Co-Thickening of Waste Activated Sludge	60 – 80 $m^3/m^2 \cdot d$	169,500 to 226,000 m^3/d
With Co-Thickening of Waste Activated Sludge	50 – 60 $m^3/m^2 \cdot d$	141,000 to 169,500 m^3/d
Notes: n/a – not available 1. Peak Day and Peak Instantaneous		

Utilization of the existing primary clarifier tankage will depend on the preferred secondary treatment alternative. The use of the existing primary clarifiers will be considered in the discussion of secondary treatment alternatives.

2.5.2 Existing Disinfection System

Primary effluent is disinfected year round by chlorination before it is discharged to the St. Lawrence River.

Chlorine contact time is provided in a chlorine contact tank with a volume of 824 m^3. The MOE Design Guidelines (2008) recommend a minimum contact time of 30 minutes at average daily flow and no less than 15 minutes at peak flow. Based on the volume of the existing chlorine contact tank, the disinfection system can only provide the minimum recommended contact time for a peak flow of 79,000 m^3/d. It should be noted that some additional contact time is provided in the outfall.

3.0 PRELIMINARY ASSESSMENT OF SECONDARY TREATMENT DESIGN OPTIONS

There are many different technologies available to provide secondary level treatment. These technologies include suspended growth processes, such as conventional activated sludge (CAS) and membrane bioreactors (MBR), and fixed-film processes such as rotating biological contactors (RBC), biological aerated filters (BAF) and moving bed biological contactors (MBBR). Each process has inherent advantages and limitations, making them suitable for different applications.

This section focuses on a preliminary assessment of secondary treatment processes to determine feasible options for implementation at the Cornwall WWTP.

3.1 Secondary Treatment Design Alternatives

Several treatment alternatives were considered for implementation at the Cornwall WWTP. To meet the new treatment objectives (**Table 2**), the existing treatment process would need to be upgraded to secondary treatment with partial nitrification capability. The following technologies were considered for upgrade of the Cornwall WWTP to biological secondary treatment.

- Activated Sludge Processes (Conventional and Extended Aeration);
- Rotating Biological Contactor (RBC);
- Moving Bed Biological Reactor (MBBR);
- Sequencing Batch Reactors (SBR);
- Membrane Bioreactors (MBR); and
- Biological Aerated Filters (BAF).

3.1.1 Activated Sludge

The activated sludge process (ASP) is one of the most widely used secondary treatment processes. There are many modifications of the activated sludge process, but all consist essentially of an aerated biological reactor (aeration tank) followed by a secondary clarifier. In the biological reactor, suspended biomass degrades the influent organic material. The biomass is subsequently separated from the wastewater in a secondary clarifier. Thickened biomass from the clarifier underflow is recycled to the aeration tank to maintain a desired biomass concentration. **Figure 2** presents a typical activated sludge process configuration.

The activated sludge process is a robust, well-proven process for treating wastewater under widely varying environmental conditions due to its operational flexibility.

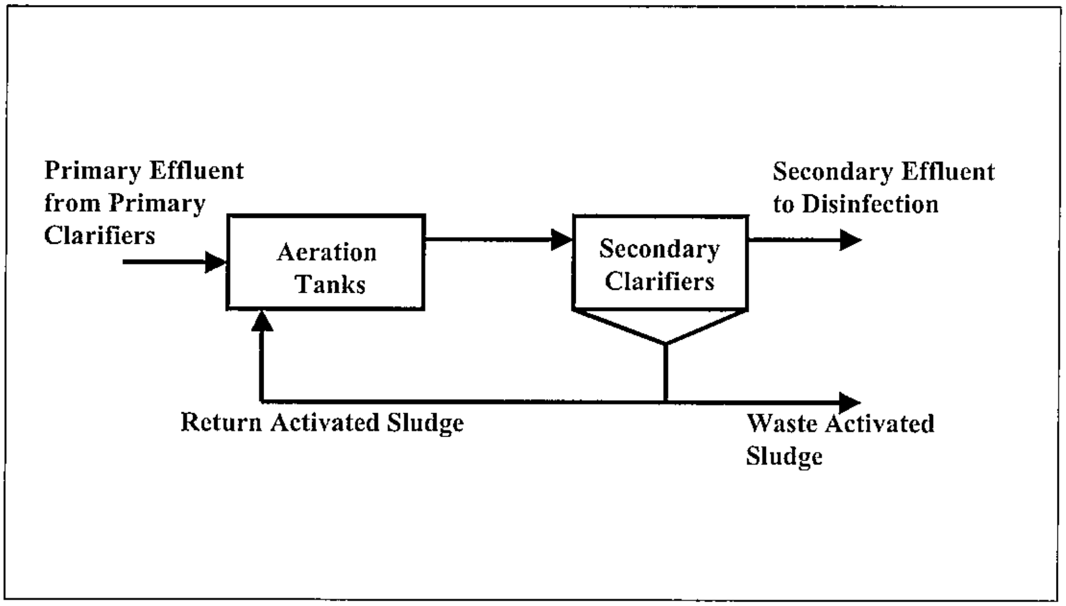

Figure 2 – Typical Activated Sludge Process Configuration

Operators can optimize the process for a given set of environmental conditions (i.e. temperature and loading variations) by varying the biomass inventory and sludge age.

3.1.1.1 Conventional Activated Sludge Process

The conventional activated sludge (CAS) process typically uses long rectangular plug-flow aeration basins, with influent and return activated sludge (RAS) introduced at one end of the basin and effluent removed at the other. Aeration is typically supplied through air diffusers or mechanical aerators. The mixed liquor suspended solids (MLSS) concentration in the aeration tanks of a conventional ASP is typically within a range of 1,000 mg/L to 3,000 mg/L without nitrification and 3,000 mg/L to 5,000 mg/L with nitrification (MOE, 2008).

The CAS process is well-suited for treating low strength domestic waste. Selection of process operating parameters is dependent on environmental factors and the desired effluent quality.

The CAS process typically removes 90 to 95 percent of the raw sewage BOD_5, and could be designed to nitrify year round provided adequate aeration tank and secondary clarifier capacity is provided. The MOE Design Guidelines (2008) identify that effluent quality from a CAS process is typically 15 mg/L for BOD_5 and 15 mg/L for TSS. The effluent ammonia concentration in a CAS process with nitrification is typically 3 mg/L. With chemical addition, total phosphorus effluent levels less than 1 mg/L are readily attained using the CAS process (MOE, 2008). Experience at other wastewater treatment facilities in Ontario has shown the secondary treatment processes with chemical addition for phosphorus removal can regularly achieve effluent TP concentrations of <0.5 mg/L. Therefore, the effluent quality would meet the anticipated secondary treatment design objectives at the Cornwall WWTP.

Ammonia removal, a process called nitrification where ammonia is converted to nitrate, within the secondary treatment process can be accomplished by operating the process at solids retention time (SRT) of 10 days or more while supplying adequate oxygen. Phosphorus removal can be accomplished through chemical addition before, during or immediately following the aeration tank. The most common addition point is immediately following the aeration tank. Chemical addition following aeration is typically more efficient than pre-precipitation (addition to primary), and thus chemical cost savings can be achieved.

A CAS process would be compatible with the primary clarifiers, chemical dosing system and anaerobic digesters at the existing Cornwall WWTP, maximizing the use of the current tankage and equipment, while minimizing the potential for significant retrofits or the need for decommissioning.

With the implementation of an ASP, it would be necessary to co-thicken the waste activated sludge (WAS) in the primary clarifiers, or to thicken the WAS separately and then blend it with primary sludge (PS) upstream of digestion. Co-thickening the WAS would reduce the theoretical capacity of the primary clarifiers.

3.1.1.2 Extended Aeration

The extended aeration (EA) process, a variation of the activated sludge process, uses longer hydraulic retention time (15 hours) in the aeration step compared to CAS. Longer HRT provides greater process resilience to shock loading and ability to treat high flows. This process is preferred for treatment of high strength wastewater, and is typically implemented following preliminary treatment, without primary sedimentation. The process is designed to operate at solids retention time of 15 days or more, and would achieve complete nitrification year round.

The MOE Design Guidelines (2008) identify that effluent quality from an EA process is typically 15 mg/L for BOD_5 and 15 mg/L for TSS. The effluent ammonia concentration in a CAS process with nitrification is typically 3 mg/L. With chemical addition, total phosphorus effluent levels less than 1 mg/L are readily attained using the EA process (MOE, 2008). Experience at other wastewater treatment facilities in Ontario has shown the secondary treatment processes with chemical addition for phosphorus removal can regularly achieve effluent TP concentrations of <0.5 mg/L. Therefore, the effluent quality would meet the anticipated secondary treatment design objectives at the Cornwall WWTP.

Upgrading to an EA process would involve significant capital costs associated with higher aeration volume requirements than the CAS process. Additionally, the EA process would not utilize the existing primary clarifiers or the anaerobic digesters. The existing primary clarifiers may be converted to equalization tanks, storage or to aerobic digesters. Conversely, the existing anaerobic digesters could be retrofitted to aerobic digesters. As a result, additional capital costs would be incurred due to the decommissioning or retrofitting of the primary clarifiers and anaerobic digesters.

3.1.2 Rotating Biological Contactor (RBC)

The rotating biological contactor (RBC) process consists of an RBC reactor and a secondary clarifier. A schematic of a typical RBC configuration is shown in

Figure 3.

The RBC is an attached growth process where biomass grows on the surface of a rotating disc, which is partially submerged in the wastewater. The rotation of the media carries a film of wastewater that contacts with air, supplying the oxygen for biological growth on the media surface. As the thickness of the biomass layer increases, it is sheared from the media and flows with the wastewater to the secondary clarifier. The RBC process is resilient to shock hydraulic and organic loads. However, since the process relies on spontaneous biomass shearing there is no effective way to control its operation and performance (final effluent quality), which in CAS is accomplished by regulating the biomass inventory. Therefore RBC's offer fewer opportunities for optimization or fine-tuning of the process and for energy savings.

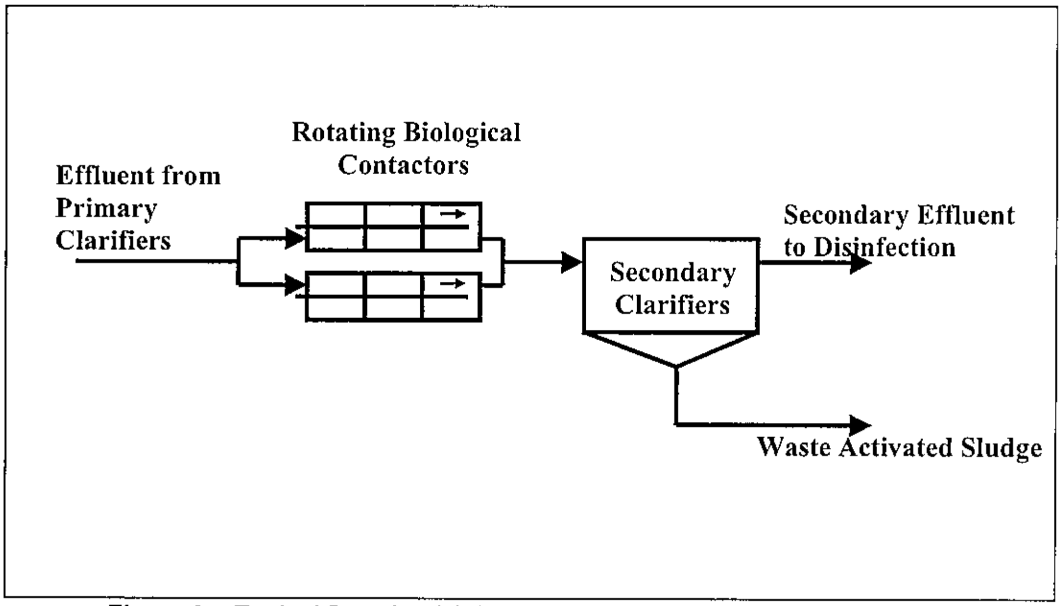

Figure 3 – Typical Rotating Biological Contactor Process Configuration

An RBC plant usually involves a number of parallel trains of RBC reactors (typically 4 to 7 RBC disks per train). Each shaft typically contains one media disc pack 8.2 m (27 ft.) in length by 3.7 m (12 ft.) in diameter (WEF, 1991). The shaft is either electrically or air driven. RBC's are usually covered in colder climates with fibre glass enclosures to help minimize heat loss.

One of the most important considerations in the design of RBC's is the first stage organic loading rate, and the overall organic loading rate. At higher first stage loadings, performance can deteriorate as a heavy biomass film forms. Heavy biomass growth can result in excessively high torque required to rotate the shaft, which can lead to premature shaft, motor and bearing failures. Odour problems have also been reported with overloaded first stage processes. Step feeding could be used to reduce the load on the first stage. Air driven RBC's are less prone to biomass build-up on the first stage due to the scouring action provided by the diffused air in the wastewater. Early RBC plants were prone to shaft and media failures. Design modifications to more recent plants have mitigated many of these issues.

The MOE Design Guidelines (2008) identify that effluent quality from an RBC process is typically 15 mg/L for BOD_5 and 20 mg/L for TSS. The effluent ammonia concentration in a CAS process with nitrification is typically 3 mg/L. With chemical addition, total phosphorus effluent levels less than 1 mg/L are readily attained using RBC's. Experience at other wastewater treatment facilities in Ontario has shown the secondary treatment processes with chemical addition for phosphorus removal can regularly achieve effluent TP concentrations of <0.5 mg/L. The biomass from the RBC process can have poor settling characteristics, potentially requiring a larger secondary clarifier surface area than a CAS process. The average effluent TSS concentration meets the anticipated design treatment limit for TSS, but would not meet the design objective of 15 mg/L.

Similar to the ASP's, chemicals can be added before the secondary clarifier for precipitation of phosphorus. This would also offer more efficient phosphorus removal and may help enhance biosolids flocculation and settleability. The WAS from an RBC process would need to be either co-thickened in the primary clarifiers prior to digestion or thickened separately.

In Ontario, RBC's are operated at the Guelph WPCP and Niagara Falls Stamford WPCP, which has approximately 50,000 m^3/d capacity. Based on recent historic (3-year average) effluent quality, the RBC's at the Niagara Falls Stamford WPCP produced average effluent $cBOD_5$ and TSS concentrations of 11 mg/L and 15 mg/L, respectively, higher maximum monthly average values are experienced.

3.1.3 Moving Bed Biological Reactor (MBBR)

The moving bed biological reactor process consists of an aeration basin filled with suspended media and a secondary clarifier. The process, which is patented by Kaldnes®, is based on the biofilm principle, and is essentially a fixed film system. A schematic of the MBBR process would be the same as that of the ASP, shown in **Figure 2**.

The core of the process is the suspended media, or biofilm carrier elements, designed to provide a large protected surface area for biofilm growth, enabling considerably higher biomass inventories to be carried in a given tankage volume compared with other ASP's. This can result in a reduction in the required footprint of the facility relative to other ASP's. The biofilm carrier elements are kept suspended in the water by air from the diffusers in aerobic reactors, and by means of mixers in anaerobic and/or anoxic reactors. Sieves or fine screens are used to retain the biofilm carrier elements in the reactor.

The process uses coarse bubble diffusion, which typically has lower oxygen transfer efficiency than fine bubble diffuser systems commonly used in ASP's. Furthermore, the manufacturer recommends a minimum dissolved oxygen (DO) concentration of 3 to 4 mg/L to ensure that the biofilm is maintained under aerobic conditions. This value is higher than the typically recommended DO concentration of 2 mg/L for ASP's. Therefore, considerably higher air flows would be required for MBBR processes, increasing the capital and energy costs associated with the blower systems. However, the higher blower costs can be partially mitigated by specifying deeper reactors to improve transfer efficiency. Additionally, the aeration energy cost would be offset by eliminating the need for return activated sludge pumping and reduced potentially aeration system maintenance cost (reduced need to replace diffusers).

The MBBR process does not require backwashing. The dead organisms fall from the carrier elements and are carried in the aeration tank effluent to the secondary clarifiers. The concentration of the aeration tank effluent is typically approximately 300 mg/L, which is considerably less than other ASP's. This results in a considerably lower solids loading rate on the secondary clarifiers relative to suspended growth systems. However, the settleability of the solids from the suspended/attached growth process is typically poorer than other ASP's. Therefore, the sizes of the secondary clarifiers for a MBBR process are typically similar to those for other ASP's.

The biomass in a suspended/attached growth process is resilient against factors such as temporary nutrient limitations, toxic slug loads, pH changes and temperature changes. Since the biomass is attached to media, which is kept in the reactor using a sieve, the MBBR process is less susceptible to solids washout during peak wet weather flows than conventional ASPs.

Sludge from the MBBR process would need to be either co-thickened in the primary clarifiers or thickened separately and blended with primary sludge prior to digestion. Co-thickening the WAS would reduce the theoretical capacity of the primary clarifiers.

Although the suspended/attached growth process has been extensively tested under different conditions in pilot and full-scale plants since 1989, there has been limited full-scale application of MBBR's in Ontario. The process has demonstrated the ability to achieve good removal of BOD_5 and nitrification even under the extreme winter climate.

Due to the elimination of RAS recycle in an MBBR system, some operational costs may be saved as a result of the reduction in pumping requirements; however, these savings are offset by the increased aeration requirements due to the lower oxygen transfer efficiency of coarse bubble aeration and the higher DO operating set-point.

3.1.4 Sequencing Batch Reactor (SBR)

A sequencing batch reactor (SBR) is a "fill-and-draw" activated sludge treatment system, where aeration and secondary clarification processes are carried out sequentially in the same tank. A schematic of the SBR process is shown in **Figure 4**.

Unlike other ASP's in which flow moves continuously along a series of tanks, the SBR is a time-oriented batch system, which can satisfy different treatment objectives by simply modifying the application and duration of mixing and aeration in a single-tank, making the SBR process very flexible. A typical operating sequence for a SBR is composed of the following five stages: fill, react (aeration), settle (mixing/aeration off to allow clarification), draw (decant) and idle. Sludge wasting is generally conducted during the settle or idle phases, but can occur in the other phases depending on the mode of operation.

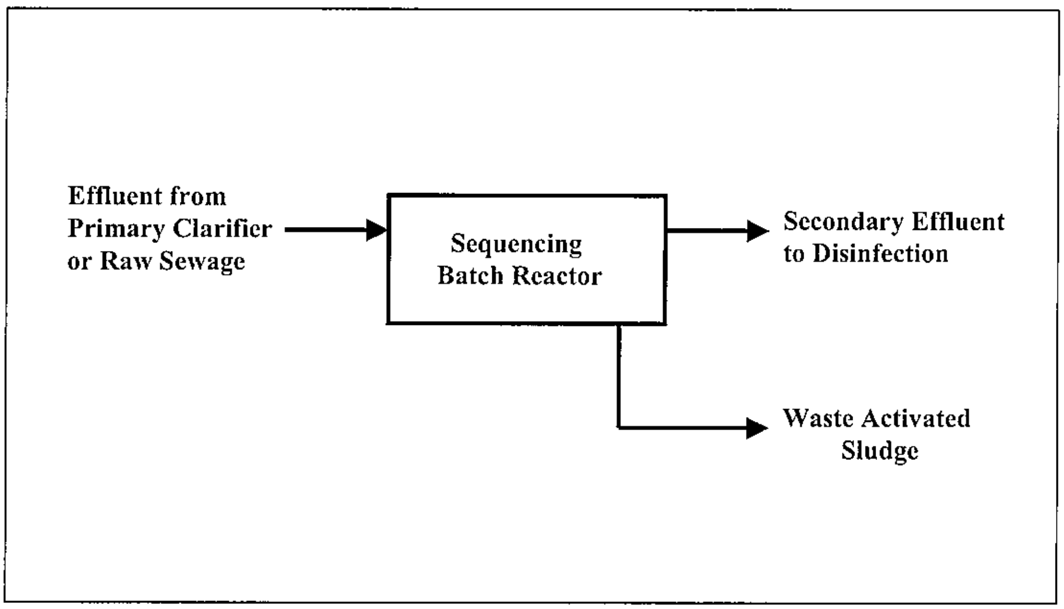

Figure 4 – Typical Sequencing Batch Reactor Process Configuration

The main advantages of the SBR system over a conventional ASP are that it has a compact footprint and requires fewer civil and mechanical works. The process incorporates more controls and significant automation compared to conventional ASP, therefore can be more complex to operate, especially for larger systems.

The SBR tank serves as an equalization basin during the fill stage. Hence, it is able to handle unsteady conditions, such as peak flows and shock loads, without significant degradation in effluent quality and without the need for additional tanks.

The BOD_5, TSS and ammonia removal performance efficiency of the SBR process is expected to be similar to that achieved in conventional ASP's. Chemical phosphorus removal can be achieved in an SBR by adding chemicals to the SBR feed. Alternatively, phosphorus removal could be achieved using specially designed SBR's equipped with biological nutrient removal (BNR) technology. The SBR process is suited to producing the cyclically alternating aerobic, anoxic and anaerobic conditions required to achieve BNR. An SBR designed for biological phosphorus removal can achieve about 1 mg/L TP. Chemicals can be added to the SBR influent to promote additional removal of TP.

The clarification process in an SBR is normally much more efficient than in conventional ASP plants because the reactor contents are under nearly quiescent conditions during the settle mode. Biological sludge production rate from the BNR process is anticipated to be similar to that of other ASP's.

The WAS would need to be either co-thickened in the primary clarifiers or thickened separately prior to digestion. If the SBR's are to be operated in a BNR mode, then co-thickening of WAS is not recommended, since the septic conditions in the primary clarifiers would promote the release of phosphorus.

Many of the disadvantages of the SBR process are related to the lack of experience with large, full-scale SBR systems. There are limited design data available, and design standards are not widely accepted or known. Equipment limitations have also been a major source of concern with SBR processes. The effluent quality depends upon a reliable decanting system and many of the difficulties experienced at existing facilities have been related to the decanting equipment. Plugging of air piping and diffusers may also occur during settle, draw and idle periods. The designer should ensure that diffusers that are resistant to fouling are specified for SBR applications. Another significant disadvantage with the SBR process is that, as the system gets larger, the sophistication of the control systems (timing units and level sensors) increases. Consequently, the SBR has been primarily used only for relatively low capacity systems. Many of these disadvantages have been resolved with recent innovations in process control equipment.

Many small SBR installations exist throughout Ontario. The largest existing SBR plant, located in Asia, has a design flow capacity of about 380,000 m^3/d.

3.1.5 Membrane Bioreactor (MBR)

Membrane bioreactors (MBR) for municipal wastewater treatment, such as GE Zenon's proprietary ZeeWeek® process, consist of a suspended growth biological reactor coupled with a microfiltration membrane system. A schematic of a typical MBR process configuration is shown in **Figure 5**.

The microfiltration membranes, which are in direct contact with the mixed liquor, effectively replace the solids separation function of the secondary clarifiers and/or granular media filters. Vacuum is applied to a header pipe connected to the membranes by a pump. This vacuum draws the treated effluent through the hollow fibre membranes (0.1 micron pore size) and into the pump, which transfers the treated effluent to disinfection. The external surface of the hollow fibres is continuously scoured using airflow introduced at the bottom of the membrane module. The airflow also provides a portion of the biological process oxygen requirements. A diffused air system is used to provide the remainder of the biological oxygen requirements. Excess biological sludge is pumped directly from the process tank.

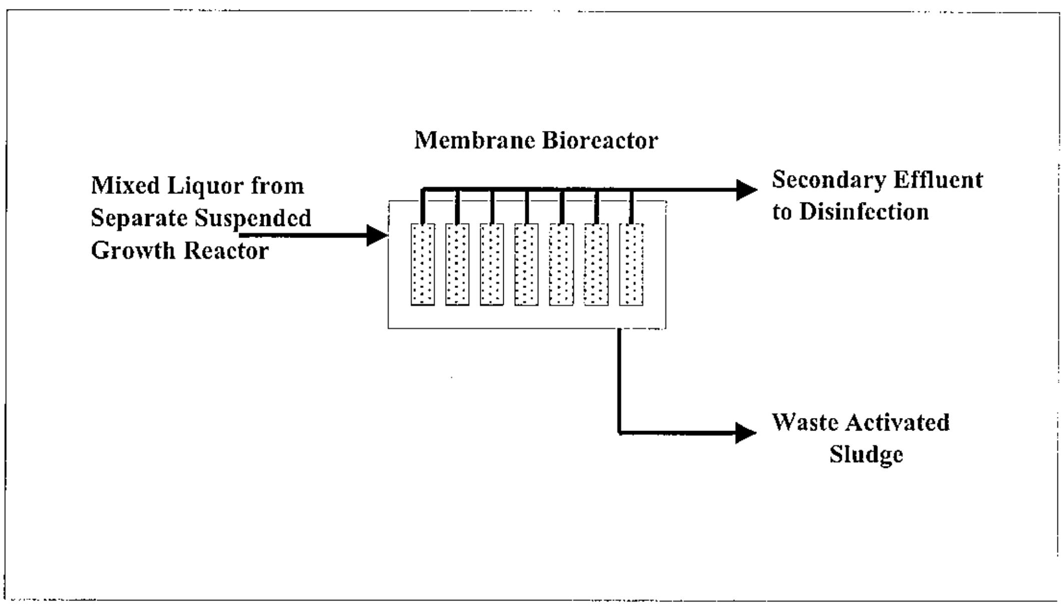

Figure 5 – Typical Membrane Bioreactor Process Configuration

The membrane bioreactor process is not limited by the sludge settling characteristics, as with other ASP's. As such, an MBR can be operated at considerably higher mixed liquor suspended solids (MLSS) concentrations (i.e. in the range of 12,000 to 20,000 mg/L) than other ASP's. The elevated biomass concentrations provide high removal efficiencies of soluble and particulate biodegradable material in the waste stream. As such, significantly lower hydraulic residence times are required for a membrane bioreactor relative to other ASP's to achieve the same F/M (food to microorganisms) ratio. It also allows the biological process to be operated at extended solids retention times (SRT's), ensuring complete nitrification even under extreme cold weather conditions. Many MBR systems are operated with SRTs exceeding 30 days. The sludge yields at extended SRTs can be considerably less than conventional ASPs due to endogenous decay of biomass in the reactor. The WAS from the MBR process would have a concentration of about 1 to 2 percent total solids (TS). It would be necessary to either co-thicken the WAS in the primary clarifiers or thicken it separately prior to digestion.

A typical MBR process achieves effluent BOD_5 and TSS concentrations of 5 and 2 mg/L, respectively, or less; and less than 2 mg/L ammonia during cold winter conditions. Phosphorus concentrations of less than 0.1 mg/L are readily achieved, with chemical precipitation.

The key disadvantages of this process are that the submerged membranes may fail while inside the reaction vessel and draw mixed liquor into the treated effluent. The operators must identify and replace the broken membrane as soon as possible by removing and inspecting each membrane module. This is an operator intensive procedure. Membrane failures could be detected by continuously monitoring the MBR effluent turbidity, or monitoring the differential pressure across each membrane module.

This technology is considered to be less proven and more operationally complex than other ASP's in municipal applications. A full-scale demonstration of the GE ZenoGem® MBR was performed at the Milton WWTP, Ontario, for treatment of primary treated effluent (Thompson, 1999). There are some small MBR's in operation in Canada. The largest facility is located in Port McNicoll, with a design ADF capacity of 1,900 m^3/d. One of the largest MBRs in operation is located in Traverse City, Michigan, with a design ADF capacity of 32,000 m^3/d (8.5 mgd).

The main factor limiting the use of this technology at larger facilities has been the high capital and operating costs. There will be an operating cost savings associated with the slightly reduced sludge production from a MBR plant relative to other ASP systems. However, the electrical costs will be significantly higher than those of the conventional ASP system due to the operation of coarse bubble and fine bubble aeration, and operation of the membrane suction pumps. There will also be additional operating costs associated with membrane replacement, which have an anticipated life about 7 to 10 years.

3.1.6 Biological Aerated Filter (BAF)

Biological aerated filters (BAF) are high-rate biological processes that utilize the features of attached growth biological filters and the efficient oxygen transfer capabilities of diffused aeration systems. The process consists of a biological reactor filled with a 2 to 5 m media bed, which serves as both a filter and a surface for biological activity. The wastewater is fed from the top or bottom of the reactor, depending on the configuration, and process air is supplied from the bottom.

The influent solids and biomass produced in a BAF accumulate in the filters and are removed by periodic backwashing using secondary effluent stored in a tank. Backwashing normally occurs once every 24 to 48 hours and uses approximately 5 percent of the treated water. Backwashing can be controlled based on elapsed time or accumulated headloss across the filter bed.

BAF processes eliminate the need for secondary clarifiers. Some additional area requirements exist for effluent storage and backwash water storage tanks. In general, the total footprint size of a BAF secondary treatment process is equivalent to approximately one third of a CAS process. BAF processes have a compact design that can be fully automated, reducing maintenance and operational requirements. They offer some operational flexibility during flow and load variations, including improved treatment of dilute and cold wastewater.

BAF processes can be configured for carbon removal, nitrification, denitrification and chemical phosphorus removal. Their modular design is an advantage for future capacity upgrades. A schematic of a typical BAF process is shown in **Figure 6**.

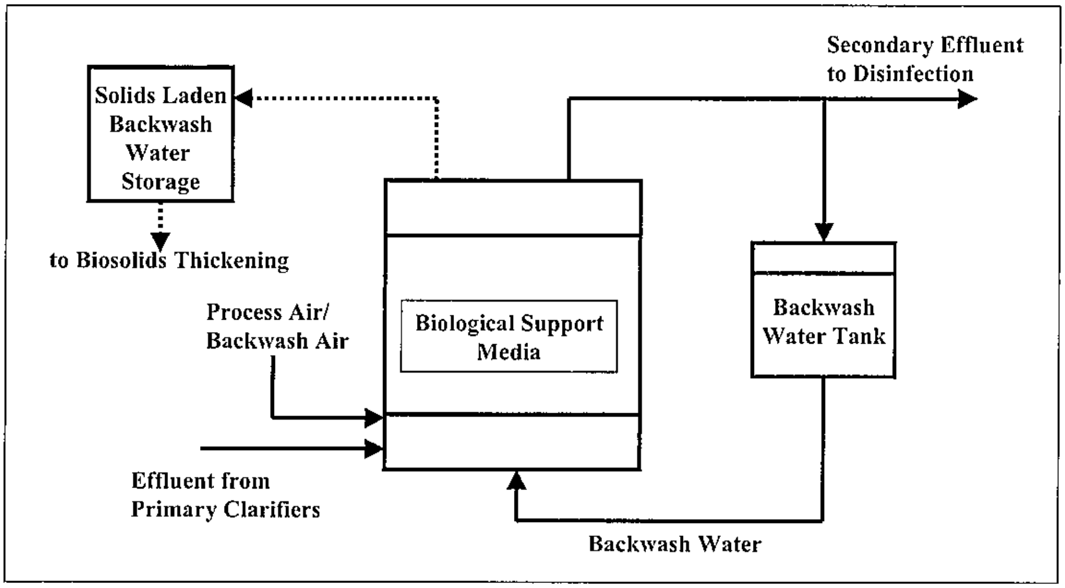

Figure 6 – Typical Biological Aerated Filter Process Configuration

There are various proprietary suppliers of the BAF technology. US Filter/Kruger developed the BIOCARBONE downflow filtration system with air introduced at the bottom of the filter. There are six BIOCARBONE facilities in Canada. This technology has experienced some odour problems and other limitations and has subsequently been replaced by the US Filter/John Meunier Inc. BIOSTYR upflow filtration system. In the BIOSTYR process, the influent wastewater and process air are applied to the bottom of the reactor. This process has been implemented at Kingston, ON as a similar secondary treatment process upgrade to a primary treatment plant, and is also being implemented at Boisbriand, QC, There are other Biostyr systems operating in the United States and Europe.

There is also the Ondeo Degremont BIOFOR upflow filtration system. There are BIOFOR facilities in Quebec, Ontario and Alberta. The Ontario locations include Thunder Bay and Windsor. The Lou Romano Water Reclamation Plant located in Windsor has a maximum design capacity of 283 ML/d at average (dry weather) flow conditions.

Effluent from BAF treatment processes have reported TSS and BOD_5 concentrations of less than 10 mg/L. Effluent ammonia concentration of less than 1.0 mg/L has been reported in a combined BOD_5 removal and nitrification process. Chemical phosphorus removal can be achieved with coagulant addition to the primary clarifiers. Coagulant could also potentially be added immediately prior to the BAF units however this is not commonly employed as there is potential for plugging of the filters and reduced filter run lengths with this operating mode.

Additionally, care must be exercised in selecting the appropriate chemical dosages to ensure that the biomass in the biological treatment process is not nutrient starved due to excess phosphorus removal.

The solids yield from a BAF operation would be similar to conventional ASP's. The backwash solids typically have excellent settling characteristics. The backwash water is settled in a separate tank, and the sludge withdrawn from the bottom of the tank can be co-thickened with raw wastewater in the primary clarifiers, or thickened separately.

The BAF can have slightly higher energy requirements compared to CAS if additional energy is needed for influent pumping through the filter. However, if gravity flow from the primary clarifiers through the BAF process can be utilized, energy requirements would be reduced. The elimination of RAS pumping in combination with gravity flow to further reduce energy requirements can potentially result in similar energy usage to that of CAS.

3.2 Summary of Advantages and Disadvantages of Secondary Treatment Design Alternatives

Table 5 presents a summary of the advantages and disadvantages of the secondary treatment design alternatives presented in the preceding sections.

Table 5 – Summary of Advantages and Disadvantages of Secondary Treatment Design Alternatives

Process	Advantages	Disadvantages
Conventional Activated Sludge (CAS)	• Proven, robust treatment process with long history of application in similar climates • Low operational complexity • Flexible process with potential for upgrade (i.e. BNR process) • Very good compatibility with existing infrastructure	• Process performance can be limited by sludge settleability • Potential for odour from primary clarifiers • Relatively large footprint
Extended Aeration (EA)	• Proven, robust treatment process with long history of application in similar climates • The most common secondary treatment process for small to medium sized WWTPs in Ontario • Low operational complexity	• Process performance can be limited by sludge settleability • Incompatibility with existing infrastructure would require decommissioning or significant retrofitting of existing tankage and equipment
Moving Bed Biological Reactor (MBBR)	• Low operational complexity • Good compatibility with existing infrastructure • Smaller footprint than CAS • Less susceptible to washout during peak wet weather flows	• Relatively new technology • No experience in Ontario • May require pilot testing prior to full-scale implementation • Higher energy costs • Sludge can possess poor settling characteristics • Little control over effluent quality under varying environmental conditions

Rotating Biological Contactor (RBC)	• Low operational complexity • Modular design – relatively simple capacity upgrades • Low energy requirements	• Inconsistent effluent quality • Limited degree of control – fewer opportunities for optimization • Limited application in Ontario for domestic wastewater treatment • Limited capacity for potentially more strenuous future effluent limits • Potential for freezing in cold climates • Biomass can possess poor settling characteristics
Sequencing Batch Reactor (SBR)	• Smaller footprint because aeration and clarification is combined in a single tank • Low capital and O & M costs • Flexible operation	• Limited application in Ontario • Limited experience in large, full-scale applications • Complex mechanical and electrical process • Difficulties may be encountered under high wet weather flows • Sensitive to industrial discharges • More complex operation than CAS
Membrane Bioreactor (MBR)	• Smaller footprint • Tertiary treatment effluent quality	• Relatively new technology • Limited experience in Ontario • May require pilot testing prior to full-scale implementation • High capital and O & M costs • Operationally complex
Biological Aerated Filter (BAF)	• Smaller footprint than CAS • May be fully automated, reducing O & M costs • Flexible operation • Modular design – relatively simple capacity upgrades	• Complex mechanical and electrical control systems • Limited application in Ontario for domestic wastewater treatment

3.2.1 Qualitative Comparison of Treatment Alternatives

The secondary treatment alternatives were assessed in a qualitative manner in order to identify feasible alternatives for the Cornwall WWTP. The criteria for screening of alternatives were developed by considering issues important in the selection of a secondary treatment process. The alternatives were assessed based on the following criteria:

- Historical application at other Ontario WWTPs;
- Process complexity/degree of training required;
- Compatibility with existing infrastructure/treatment processes;
- Degree and consistency of treatment performance;
- Potential for optimization/increased capacity and treatment requirements.
- Potential capital costs;
- Potential O & M costs;
- Ability to meet more stringent (potential future) effluent criteria;
- Modular design/expansion potential; and
- Potential site and neighbouring community impact.

The scoring for each of the screening criterion were assigned to each technology based on a relative comparison of the technologies. **Table 6** presents a preliminary comparison assessment of the secondary treatment alternatives. It should be noted that the rankings presented in **Table 6** are preliminary only, and are subject to further review.

Table 6 – Preliminary Secondary Treatment Design Alternative Scoring

Criteria	CAS	EA	MBR	SBR	RBC	BAF	MBBR
Historical Application	5	5	2	3	2	2	1
Ease of Operation	5	5	1	3	5	5	4
Compatibility w/Existing Infrastructure	5	1	2	3	5	5	3
Degree and Consistency of Treatment Performance	3	3	5	3	2	4	2
Potential for Optimization	5	5	5	4	1	4	2
Potential Capital Costs	3	2	1	3	3	3	3
Potential O & M Costs	3	3	1	3	4	3	3
Ability to Meet More Stringent Effluent Criteria	3	3	5	3	3	4	3
Modular Design/Expansion Potential	4	4	5	3	4	5	3
Potential Site and Neighbouring Community Impact	3	3	5	4	3	4	3
Total [1]	39	34	33	32	32	39	27

Notes:

Criteria Assessment based on a scale of 1 – 5. A rating of 5 is most favourable and a rating of 1 is least favourable.

1. Total is the sum of the ratings for each category.

Based on the screening criteria, the preliminary preferred secondary treatment upgrade alternatives are CAS or BAF.

CAS is a proven secondary treatment process with a long history of application in Ontario. The process has ability to meet effluent objectives in combination with simple and flexible operation. Upgrade of the existing treatment process at the Cornwall WWTP to CAS could utilize the existing infrastructure, maximizing the use of current tankage and equipment and minimizing capital costs.

BAF processes provide exceptional effluent quality, with low land requirements. In a BAF, biological treatment and solids separation occurs in a single unit process rather than two unit processes. Upgrade of the existing treatment process at the Cornwall WWTP to BAF could utilize the existing infrastructure, maximizing the use of current tankage and equipment and minimizing capital costs.

Based on the preliminary evaluation criteria, both treatment alternatives are feasible for upgrade at the Cornwall WWTP. Further client input and possibly criterion weighting considerations are suggested to confirm the preferred treatment alternative.

Preliminary sizing is provided for both CAS and BAF in the following section to facilitate the development of preliminary conceptual plans and capital costing of each option based on Peak Day and Peak Instantaneous flow of 160,000 m^3/d. However, this will be reviewed further during the design phase based on value engineering and a comprehensive cost-benefit analysis. In any event, the peak flow design basis shall be 130,000 m^3/d, as a minimum, consistent with the 2005 ESR.

3.3 Preliminary Sizing of Short-Listed Secondary treatment Alternatives

3.3.1 Conventional Activated Sludge

Preliminary design requirements for the bioreactors were developed based on the following assumptions:

- Year-round nitrification is to be provided;
- Three pass aeration tanks would be constructed to achieve plug flow for better nitrification kinetics and to provide the capability to perform step-feed or biological nutrient removal;
- Operating MLSS of 4,000 mg/L; and,

- WAS generation rate would be approximately 1.0 kg WAS/kg BOD_5 removed to account for sludge yield and sludge generation due to chemical addition for phosphorus removal.

Preliminary design requirements and design operating conditions for the bioreactors are presented in **Table 7**.

Table 7 – Bioreactors – Preliminary Design Requirements and Operation Conditions

Parameter	Value	Typical Design Value
Bioreactor		
Number	4	n/a
Volume (each)	2,944 m^3	n/a
Footprint (each)	640 m^2	n/a
Volume (total)	11,776 m^3	n/a
Footprint (total)	2,560 m^2	n/a
Average BOD_5 Loading[1]	4,703 kg/d	n/a
Operating MLSS	4,000 mg/L	3,000 to 5,000 mg/L
OLR	0.40 kg $BOD_5/m^3 \cdot d$	0.31 to 0.72 kg $BOD_5/m^3 \cdot d$ [4]
F/M_v[2]	0.15 d^{-1}	0.05 to 0.25 d^{-1} [4]
SRT[3]	10 d	> 10 d at 5°C [4]
HRT	4.3 h	> 6 h [4]

Notes:

MLSS – mixed liquor suspended solids

OLR – organic loading rate

F/M_v – food to microorganism ratio

SRT – solids retention time

HRT – hydraulic retention time

1. Based on an a BOD_5 removal of 35 percent across the primary clarifiers based on typical design values (MOE, 2008).
2. Based on the historic MLVSS:MLSS ratio of 0.66.
3. Based on the estimated WAS generation rate of 1 kg WAS/kg BOD_5 to account for solids generated by chemical addition for phosphorus removal.
4. MOE (2008).

As shown in **Table 7**, four new three-pass bioreactors would be required to treat the design flows at the Cornwall WWTP. At the design flows, the HRT would be less than the recommended value of 6h (MOE, 2008). However, it is more critical to maintain the operating parameters of SRT and F/M_v within MOE Guidelines. Because SRT and F/M_v are dependent on the biomass inventory that can be maintained in the secondary treatment process, the maximum bioreactor capacity is dependent on secondary clarifier hydraulic and solids loading capacity, therefore, it is likely possible to operate the bioreactors at a HRT less than 6 hours.

Preliminary upgrade requirements for the secondary clarifiers were developed based on the following assumptions:

- Chemical addition upstream of the secondary clarifiers for phosphorus removal; and,
- The bioreactors would be operated at the design MLSS concentration of 4,000 mg/L.

Preliminary design requirements and design operating conditions for the secondary clarifiers are presented in **Table 8**.

Table 8 – Secondary Clarifiers – Preliminary Design Requirements and Operation Conditions

Parameter	Value	Typical Design Value
Secondary Clarifier		
Number	4	n/a
Surface Area (each)	1,330 m^2	n/a
Surface Area (total)	5,320 m^2	n/a
Peak SOR	30 $m^3/m^2 \cdot d$	< 37$m^3/m^2 \cdot d$ [1]
Peak SLR	169 $kg/m^2 \cdot d$	< 170 $kg/m^2 \cdot d$ [1]
Notes: SOR – surface overflow rate SLR – solids loading rate 1. MOE (2008).		

As shown in **Table 8**, four new secondary clarifiers would be required to treat the design flows at the Cornwall WWTP.

3.3.2 Biological Aerated Filters

Preliminary design requirements for the biological aerated filters were developed based on the following assumptions:

- Co-thickening of backwash water would be practiced, increasing the required BAF capacity;
- TP reduction would be achieved primarily through coagulant dosing to the primary clarifiers, reducing the BOD_5 and TSS loading to the BAF units; and,
- Redundancy would be built into the design of the BAF system (N-1 conditions).

Estimated primary effluent quality based on supplier design software is presented in **Table 9**.

Table 9 – Estimated Primary Effluent Characteristics

Parameter	Primary Influent	Estimated Primary Effluent[1]
BOD_5	110 mg/L	72 mg/L
TSS	165 mg/L	70 mg/L
TKN	26 mg/L	26 mg/L
TP	4.0 mg/L	2.4 mg/L

Notes:

1. Estimated based on assumed primary clarifier percent removal performance. Note that primary effluent TP concentration is subject to further chemical removal to balance achieving plant effluent TP requirements with overall treatment process, while ensuring adequate soluble phosphorus is available for bacterial growth within the biological aerated filters.

Two proprietary processes were considered for the BAF system. Information regarding the BIOSTYR BAF by John Meunier Inc. and the BIOFOR BAF by Degremont Ltd. are presented below.

3.3.2.1 BIOSTYR BAF by John Meunier Inc.

The BIOSTYR system is an up-flow biological aerated filter treatment process. The media consists of specially treated polystyrene beads. Preliminary sizing requirements for a BIOSTYR BAF system for the Cornwall WWTP (based on conceptual design information provided by John Meunier Inc.) are presented in **Table 10**.

Table 10 – BIOSTYR Biological Aerated Filters – Preliminary Sizing Requirements

Parameter	Design Value
Filter Cells	
Cells Required	7 [1]
Surface Area (each)	147 m^2
Filter Surface Area (total)	1,029 m^2
Note: 1. Based on a Peak Day and Peak Instantaneous Flow of 160,000 m^3/d. John Meunier has advised that the number of cells required can be reduced to six to achieve a Peak Day and Peak Instantaneous Flow capacity of 130,000 m^3/d	

As shown in **Table 10**, six to seven BIOSTYR BAF cells, each having a surface area of 147 m^2, will be required in order to treat Peak Day and Peak Instantaneous flows of 130,000 to 160,000 m^3/d respectively. John Meunier has advised that a 6 BIOSTYR BAF cells are required to treat the anticipated organic loading at an Average Day Flow of 65,318 m^3/d In order to be conservative and for operational flexibility, the number of cells could be increased by one additional cell for redundancy. This would allow for the plant to maintain its capacity in the event that one cell may be out of service. However, John Meunier has advised that adding a redundant cell is not required to obtain a process guarantee as this is entirely an Owner decision. The number of cells and peak flow capacity to be provided will be reviewed further during design based on value engineering and a comprehensive cost-benefit analysis.

Each cell is designed for backwashing a maximum of once per day. Each backwash cycle uses 1,286 m^3 of water, for a maximum of 9,003 m^3 of backwash water recycled to the primary clarifiers based on seven cells in operation. Additionally, a backwash storage/equalization tank is recommended to handle the large flows that are generated over short periods during the backwash cycles. The volume recommended by the BAF supplier is 2,400 m^3, or the volume of approximately two backwash cycles.

3.3.2.2 BIOFOR BAF by Degremont Ltd.

The BIOFOR system is also an up-flow biological aerated filter treatment process with first stage BIOFOR C cells installed in series with second stage BIOFOR N cells to facilitate the removal of carbonaceous and nitrogenous contaminants. The media consists of biolite. The BIOFOR C cells operate in series with the BIOFOR N cells during average day conditions. However, the BIOFOR C cells operate in parallel with the BIOFOR N cells during high wet-weather flow periods.

Preliminary design requirements and design operating conditions for the BIOFOR BAF system (based on conceptual design information provided by Degremont Ltd.) are presented in **Table 10**.

Table 11 – BIOFOR Biological Aerated Filters – Preliminary Sizing Requirements

Parameter	Design Value
Filter Cells	
BIOFOR C Cells required	6 @ 72 m^2
BIOFOR N Cells required	9 @ 81.75 m^2
Filter Surface Area (total) [1]	1,167.75 m^2
Note: 1. Degremont has advised that the same number of cells and filter surface area is required for a Peak Day and Peak Instantaneous Flow capacity of 130,000 or 160,000 m^3/d.	

4.0 REVIEW OF DISINFECTION ALTERNATIVES

There are several disinfection technologies available, including chlorine, chlorine dioxide, ozone, bromine, bromine chloride, iodine, UV radiation and gamma ray irradiation; however, many of these technologies are not utilized at wastewater treatment plants due to the high capital/operating costs and complex maintenance and operation. Chlorination and UV radiation are the most commonly used technologies in smaller scale facilities in North America. Therefore, based on the available knowledge and experience, these technologies were selected for a detailed review for the Cornwall WWTP.

The Cornwall WWTP presently uses gaseous chlorine to disinfect its primary treatment plant effluent. The following two options were considered to meet the anticipated secondary effluent disinfection requirements at an upgraded Cornwall WWTP:

Alternative 1: Chlorination/Dechlorination

Alternative 2: UV disinfection.

The following sections present a discussion of the disinfection options considered for application at the Cornwall WWTP. For both options, it was assumed that the upgraded plant will need to provide year round disinfection.

4.1 Alternative 1: Chlorination / Dechlorination

The most commonly used disinfectant in wastewater treatment is chlorine. Chlorine is relatively inexpensive and an efficient biocidal agent. Dosage and effective contact time are the two important control parameters used to optimize the chlorine disinfection process. According to the MOE Design Guidelines for Sewage Works (MOE, 2008), a contact time of 30 minutes at average dry weather flow, and 15 minutes at peak flow, with a minimum chlorine residual of 0.5 mg/L are recommended for effective disinfection of secondary effluent. It should be noted that when calculating the contact time, it is important to consider the dimensions of the contact chamber, since the length-to-width ratio will influence the effective contact time and thus the disinfection effectiveness. Also, when using chlorine, the disinfection effectiveness is greatly enhanced by effective mixing of the wastewater and chlorine.

The required chlorine dose will vary with the type of organisms present and their concentration, as well as the presence of other impurities, which can hinder the effects of the disinfection process. Typically, a dose of 2 to 9 mg Cl_2/L is required for disinfection of secondary effluent, and 1 to 6 mg Cl_2/L for disinfection of tertiary effluent (MOE, 1984). Jar tests are typically required in order to more precisely define the required chlorine dose, residual and contact time for the specific effluent characteristics at each plant.

In order to produce a non-toxic effluent prior to discharge, any remaining chlorine residual will need to be neutralized by the addition of a dechlorinating agent such as sulphur dioxide or sodium bisulphite. The dechlorination reaction occurs almost instantaneously, and typically, if approximately 30 seconds of contact time are available in the effluent piping/channels, no additional tankage is required; however, mixing at the addition point should be provided. Typically, the chlorine residual prior to and following dechlorination is continuously monitored, and used to control the dechlorination chemical dose.

Chlorine can be applied as:

- sodium hypochlorite liquid; or
- chlorine gas.

For larger treatment plants, where larger quantities of chlorine are required, gaseous chlorine is often used to reduce chemical cost. There are, however, health and safety concerns associated with the use of chlorine gas, primarily related to the transport and handling of chlorine gas cylinders, and the potential of a toxic gas leak, which could affect the neighbouring population.

For smaller facilities, the use of liquid sodium hypochlorite is generally preferred. Sodium hypochlorite is considered a hazardous material and requires proper storage containers with containment. This solution also requires care when handling.

Chemicals commonly used for dechlorination are:

1. sulphur dioxide gas; and
2. sodium bisulphite.

Sulphur dioxide in gaseous form, is applied to the chlorinated water at a ratio of 1.1 mg of SO_2 per 1.0 mg of free chlorine (White, 1986). Safety concerns with the use of sulphur dioxide gas, related to cylinder transport, handling, and the necessity for ventilation should be considered. The sulphur dioxide gas is toxic; therefore, in the event of a leak the gas could disperse and present a health and safety risk to the plant's operating staff and the neighbouring population.

Sodium bisulphite is supplied in liquid form, typically as a 38% $NaHSO_3$ solution, and has the advantage of reduced safety concerns associated with handling and health and safety risks in the event of a spill, as compared to sulphur dioxide gas. Sodium bisulphite is dosed at a ratio of 1.7 mg of $NaHSO_3$ per 1.0 mg of chlorine residual (Metcalf & Eddy, 2003).

Sodium bisulphite is slightly more expensive than sulphur dioxide; however, on the basis of the reduced health and safety risks associated with chemical storage and handling, sodium bisulphite is recommended as the preferred dechlorinating agent for the Cornwall WWTP, if chlorination is practiced.

4.1.1 Chlorine Contact Chamber

As discussed in Section 2.5.2, the existing chlorine contact tank has a volume of 824 m^3 and provides 15 minutes of contact time for peak flows of 79,000 m^3/d. Additional contact time is provided in the outfall.

In order to provide sufficient contact time at the design peak flow of 160,000 m^3/d, an additional 843 m^3 of chlorine contact tank volume would be required.

The dechlorination reaction occurs almost instantaneously. Dechlorination contact time can be provided in the outfall or in a separate dechlorination tank.

One on-line chlorine residual analyzer would be installed prior to dechlorination, and one chlorine residual or sulphite analyzer at after dechlorination. Chemical addition would be flow paced, and adjusted based on the measured chlorine and/or bisulphite residuals.

It should be noted that reuse and expansion of the existing chlorine contact tank by gravity with implementation of secondary treatment will not be possible due to hydraulic grade line and site layout considerations. A new 1,667 m^3 chlorine contact tank will need to be constructed downstream of the new secondary treatment process should chlorination/dechlorination be the preferred disinfection alternative.

4.2 Alternative 2: UV Disinfection

Ultraviolet (UV) disinfection is an alternative to chlorination/dechlorination that also produces a non-toxic effluent. It is a compact and well proven technology. Commonly, UV disinfection utilizes either low pressure lamps that emit near monochromatic UV light at a wavelength of 253.7 nm, or medium pressure lamps that emit energy at wavelengths from 180 to 1,370 nm. Medium pressure lamps have the advantage that fewer lamps are required to achieve an equivalent level of disinfection relative to low pressure lamps. However, the frequency of cleaning and maintenance, and power requirements are higher with medium pressure lamps. For the purposes of this options analysis, UV disinfection systems utilizing low pressure UV lamps were considered.

UV radiation, when compared to chlorination, is a safer method of disinfection since it is a physical disinfection process, and its use eliminates the handling, transportation, and storing of toxic, hazardous, and corrosive chemicals. UV disinfection requires minimum space; however, from the standpoint of operation and maintenance is more complex and requires skilled personnel. The energy requirements are also significant when compared to chemical disinfection methods.

The process design elements critical to effective UV treatment design are: flow rate, plant hydraulics, UV transmittance (or absorbance), and wastewater characteristics. UV treatment systems are designed for the peak plant design flow.

The UV transmittance (UVT) is related to the presence of organic and inorganic compounds, and other particles in the water that absorb and scatter light. As such, the UVT of the water influences the UV demand, and affects the sizing of a system and possibly the configuration (spacing) of the lamps. Typically, UV system suppliers assume a UVT in the 45 – 65 percent range for municipal sewage plant effluents, depending on whether the wastewater receives primary or secondary treatment.

The Cornwall WWTP accepts leachate from five landfills in the City of Cornwall. Based on historic flow data, leachate represents less than 2 percent of the flow to the plant, and flows are expected to decrease in the future. The presence of non-biodegradable organics in the plant effluent may impact the UVT of the effluent, impacting on the efficiency of UV disinfection. In a previous study (Hydromantis, 2003). UVT testing was conducted indicating the feasibility of the implementation of UV disinfection at the Cornwall WWTP.

Wastewater quality parameters critical to UV systems include hardness, manganese and iron concentrations and initial coliform density. The total hardness, manganese and iron concentrations of the water are indicators of the potential for fouling of the UV lamps. Cleaning systems are available to reduce the potential for lamp fouling in waters with high hardness and iron concentrations. Automated mechanical/chemical-cleaning systems that eliminate the need for manual sleeve wiping are available.

UVT and other water characteristics, such as hardness, manganese and iron concentrations, coliform counts, will need to be confirmed for the Cornwall WWTP effluent at preliminary design stage. Furthermore, collimated beam testing to determine the UV dose required under peak flow conditions at the Cornwall WWTP would need to be carried out during the pre-design phase.

4.3 Preferred Disinfection Alternative

The results of the 2005 ESR indicated UV disinfection as the preferred disinfection alternative for the Cornwall WWTP. The results of this review of the disinfection alternatives are in agreement with the 2005 ESR. UV disinfection is recommended as the preliminary preferred design alternative. Although chlorination/dechlorination is a proven method for disinfection of wastewater effluent, the technology can be complex and presents a hazardous risk for humans (i.e. handling, transport and storage) and aquatic life. UV disinfection is generally replacing chlorination as the preferred option for disinfection in Ontario due to the lack of disinfectant residual and lower hazardous material handling implications.

5.0 SUMMARY

Based on the results of a preliminary evaluation of liquid treatment train alternatives, both conventional activated sludge (CAS) and biological aerated filters (BAF) were identified to be the top ranked feasible alternatives for the upgraded secondary treatment process at the Cornwall WWTP. Both alternatives are compatible with the existing unit processes at the current facility and would maximize the use of existing tankage while minimizing the need to retrofit existing equipment. The suitability of both technologies was compared and discussed in detail during a Value Engineering workshop help with Cornwall staff on November 6, 2009. It was determined that because of similar life cycle costing, comparable capital costs, and a more automated system and significantly smaller footprint for BAF, that the BAF technology best addresses the needs of the City of Cornwall, consistent with the preferred secondary treatment technology identified in the 2005 ESR. It was recognized that the CAS process may offer more operational parameters that are adjustable for operators, but this, in the opinion of operators and management for the City of Cornwall, is outweighed by the automated and efficient operation of the BAF process. It was also noted that the BAF process may be more "robust" than CAS during hydraulic loading fluctuations. The BAF supplier, Maximum Daily Flow and Peak Instantaneous Flow design capacity and number of cells required will need to be reviewed and confirmed during design while considering life-cycle costs (including supply and installation costs) and other implementation, operational and maintenance considerations.

It should be noted that both John Meunier and Degremont have confirmed that piloting of their respective technology is not required for this project given the parallel piloting of both the BIOSTYR and BIOFOR technologies carried out in 2004 for similar influent characteristics at the Ravensview WWTP in Kingston. Both suppliers have confirmed that they will provide the necessary Performance Guarantee for the Cornwall WWTP project without piloting. This will enable Cornwall to save approximately $300,000 in $500,000 in piloting costs in addition to saving potential escalation costs associated with a 6-month timeframe delay required to undertake piloting.

Ultraviolet disinfection was recommended as the preliminary preferred disinfection alternative due to the simple operation and minimal health risk posed to operations staff and the environment. This recommendation is consistent with the preferred disinfection technology identified in the 2005 ESR. Sizing of the UV disinfection system should be confirmed based on the results of collimated beam testing conducted during the pre-design phase.

REFERENCES

Hydromantis, 2005. Environmental Study Report: Cornwall Waste Water Treatment Plant.

Metcalf & Eddy, 2003. Wastewater Engineering – Treatment and Reuse.

MOE, 2008. Design Guidelines for Sewage Works.