D R A F T
PHASE 3 REPORT
TOWN OF MISSISSIPPI MILLS
ALMONTE WARD COMMUNAL SEWAGE SYSTEM
CLASS ENVIRONMENTAL ASSESSMENT

August 2006

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PHASE 3 REPORT
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1.0 INTRODUCTION

1.1 Background

The Town of Mississippi Mills (Town) is an amalgamated municipality of three Wards - Almonte Ward, Ramsay Ward and Pakenham Ward. The latter two are predominately rural and serviced primarily by private wells, septic systems and holding tanks. The Almonte Ward is predominately urban and serviced by a communal water distribution system and a sewage collection and treatment system.

The communal sewage system consists of a gravity piped collection system, five sub-area pumping stations, one main pumping station and a four-cell facultative treatment lagoon system (Lagoon). The current serviced population is approximately 4,650.

The Lagoon is reaching the end of its intended service life, evidenced by berm failures, potential leakage, sludge accumulation and variable treated effluent quality. In addition, the volume of sewage generated by the current serviced population is approaching the Lagoon's rated capacity and will be insufficient to accommodate the projected 2026 population of 8,127.

The Town continues to invest in sewer separation and upgrades to the collection system. While this has reduced extraneous flows, there have been occasions when it has been necessary to bypass the treatment system as a result of a combination of high flows and insufficient pumping station capacity. The existing main Pumping Station is nearing the end of its intended service life and requires upgrading. Continuing improvements to the collection system can also be undertaken to further reduce extraneous flows and renew infrastructure.

J.L. Richards & Associates Limited (JLR) was retained by the Town of Mississippi Mills in November 2004 to assist in the completion of a Schedule 'C' Class Environmental Assessment (Class EA) that would identify problems with the existing communal sewage system and evaluate alternative solutions to meet current and future needs.

Phase 1 summarized existing background information and identified the problems associated with the existing system.

Phase 2 of the Class EA process involved the completion of several tasks, including the determination of effluent criteria and preparation of a Receiving Water Assessment; an evaluation of the existing lagoon system, including Geotechnical, Hydrogeological and Natural Environmental Investigations; an Assessment of septage receiving and potential impacts on wastewater flow and loading; public consultation planning; and an examination of alternative solutions. A Phase 2 Preliminary Findings Report and subsequent Technical Memorandum were prepared. A Mechanical Treatment Plant with Tertiary Treatment was identified as the preferred solution for the treatment related problems in consultation with the Technical Advisory Committee (TAC) for the project, as well as the public.

Phase 3 of the Class EA process is intended to identify alternative design concepts, evaluate options and determine a preferred design. Figure 1 illustrates the progress of this Class EA to date.

1.2 Summary of Phase 2 Public Consultation

Table 1 provides a summary of the Technical Advisory Committee (TAC) meetings held during Phase 2, including the purpose of the meeting and the Stakeholders that participated (refer to Appendix 'A' for Meeting Minutes).

Table 1: Summary of Technical Advisory Committee Meetings

MEETING	PURPOSE	STAKEHOLDERS
TAC Meeting No. 3 June 15, 2005 Mississippi Mills, ON	Review additional investigations including the Receiving Water Assessment, Geotechnical/ Hydrogeological Investigation and Septage Receiving.	• Attendance list unavailable
TAC Meeting No. 4 August 25, 2005 Mississippi Mills, ON	Review comments received from Open House No. 1, Draft Receiving Water Assessment Report and next steps in the planning process.	• Town of Mississippi Mills • Ministry of the Environment • Mississippi Valley Conservation Authority • Ontario Clean Water Agency • J.L. Richards & Associates Limited •
Special Meeting No. 1 September 20, 2005 Kingston, ON	Focus group meeting to address proposed MOE effluent criteria	• Town of Mississippi Mills • Ministry of the Environment • J.L. Richards & Associates Limited
TAC Meeting No. 5 November 30, 2005 Mississippi Mills, ON	Discuss proposed MOE effluent criteria to be carried forward during the Class EA and discuss impact on preferred solutions	• Town of Mississippi Mills • Ministry of the Environment • Mississippi Valley Conservation Authority • Ontario Clean Water Agency • J.L. Richards & Associates Limited
TAC Meeting No. 6 November 30, 2005 Mississippi Mills, ON	Provide information, in the form of a presentation on engineered wetlands	• Town of Mississippi Mills • Ministry of the Environment • Mississippi Valley Conservation Authority • Ontario Clean Water Agency • J.L. Richards & Associates Limited • Jacques Whitford
TAC Meeting No. 7 May 18, 2006 Mississippi Mills, ON	Discuss and review revised Phase 2 Options	• Town of Mississippi Mills • Ministry of the Environment • Mississippi Valley Conservation Authority • Ontario Clean Water Agency • J.L. Richards & Associates Limited

Two Public Consultation Meetings were held during Phase 2, as follows (refer to Appendix 'B' for documentation related to the Public Meetings):

- June 23, 2005: Open House No. 1

Following the Public Meeting, two comments were received from residents. The first comment requested further investigation into the risk of potential aerosols produced by the preferred solution. A response was prepared confirming that the preferred solution is in accordance with all known applicable guidelines and regulations (e.g., minimum separation distances). The second comment requested further information on the decision-making matrices used during Phase 2 to evaluate the alternative solutions and clarification on the Scope of Study. A response was issued addressing the decision rationale and Scope of Study concerns. No further correspondence was received on these matters.

- June 1, 2006: Public Presentation No. 2

No written comments were received as a result of the Public Meeting, although several questions were answered during the meeting.

Project documentation has been made available for public viewing on the Town website, www.mississippimills.ca, and at the Municipal Offices.

1.3 Objectives

The objectives of this Report are to:

- summarize the key information developed from Phases 1 and 2, specifically as it affects Phase 3 of the Class EA;
- identify and evaluate alternative design concepts for the preferred solution that was determined in Phase 2 of the Class EA; and
- identify preferred concepts, technologies and preliminary design information.

2.0 DESIGN BASIS AND CONSTRAINTS

Phases 1 and 2 of the Class EA included the following activities:

- determination of the design flows and organic loading characteristics of the raw sewage, including impacts from potential septage addition;
- confirmation of effluent criteria based on a Receiving Water Assessment and input from MOE Regional Technical Services (Kingston);
- evaluation of the existing geotechnical, hydrogeological and natural environments for the existing lagoon; and
- a review of possible options and selection of a preferred solution.

2.1 Raw Sewage Design Flows and Organic Loading

The raw sewage design flows were derived using historical data, the 2026 projected design population of 8,127 and applicable design guidelines. Table 2 presents a summary of the design influent flows determined during Phases 1 and 2.

Table 2: Design Influent Flows

PARAMETER	DESIGN VALUE
Average Day	4,500 m^3/day
Maximum Day	9,000 m^3/day
Peak Instantaneous	18,000 m^3/day[1]

[1] Instantaneous historical flow data is not available and a peaking factor of 4 has been assumed based on similar size and types of communities.

Table 3 outlines raw sewage quality characteristics. These characteristics were based on available historical and typical sewage concentrations and the proposed average day flow.

Table 3: Design Influent Wastewater Characteristics

PARAMETER	INFLUENT CONCENTRATIONS (mg/L)[1]	DESIGN INFLUENT LOADING (kg/day)
Ammonia	25	113
Total Phosphorus	4	18
Total Solids	720	3240
Total Suspended Solids	190	855
Volatile Suspended Solids	160	726
BOD_5	115	518

1 Average values for 2002 to 2004 used for BOD_5, total phosphorus, total suspended solids. Typical values for medium strength sewage used for parameters with no historical data (i.e., total ammonia, total solids, volatile solids) (Metcalf and Eddy, 2003).

The impacts from the potential of adding hauled septage to the raw sewage stream were also investigated. Septage volumes from the rural areas of the Town of Mississippi Mills were estimated based on available information, and the amount of septage that could be expected was calculated. A possible daily flow rate of septage addition was determined using established guidelines and data from previously completed Studies (Trow, 2003). Table 4 provides a summary of the predicted impacts that septage addition could have on the design influent flows and wastewater characteristics.

Table 4: Design Influent Wastewater Characteristics with Septage Addition

SEPTAGE FLOW 30 m^3/day	Septage Conc. (mg/L)[1]	Raw Sewage Conc. (mg/L)[2]	Mixed Conc at 100% Avg. Day (4,500 m^3/day) Sewage Flow (mg/L)	Mixed Conc at 50% Avg. Day (2,250 m^3/day) Sewage Flow (mg/L)	Typical "Medium Strength" Untreated Domestic Wastewater (mg/L)[3]	Typical "High Strength" Untreated Domestic Wastewater (mg/L)[3]
Total Ammonia	150	25	26	27	25	45
Total Phosphorus	250	4	6	7	7	12
Total Solids	40,000	720	981	1,238	720	1,230
Total Suspended Solids	15,000	190	288	385	210	400
Volatile Suspended Solids	10,000	160	225	290	160	315
BOD_5	7,000	115	161	206	190	350
Grease	8,000	90	142	194	90	100

1 Ref. Recommended Standards for Wastewater Facilities. 2004. Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers.

2 Average values for 2002 to 2004 used for BOD5, total phosphorus, total suspended solids. Typical values for medium strength sewage used for parameters with no historical data (total ammonia, total solids, volatile solids, grease) (Metcalf and Eddy. 2003. Wastewater Engineering. Pg. 186.

3 "Medium strength" and "high strength" values taken from Metcalf and Eddy. 2003. Wastewater Engineering. Pg. 186.

Based on estimated design concentrations, the results outlined in the above Table illustrate that septage addition, at a flow rate of 30 m^3/day, would significantly increase the strength of the sewage, however, it could still be handled in a typical municipal sewage treatment plant, provided adequate facilities are available, and equipment and tankage are appropriately sized.

2.2 Effluent Flows and Quality

Design effluent flows were determined using historical data and several year-round discharge scenarios. Table 5 presents the design effluent flows proposed for the mechanical treatment plant.

Table 5: Design Effluent Flows

PARAMETER	DESIGN VALUE
Average Day	4,500 m^3/day
Maximum Day	9,000 m^3/day

Effluent design objectives and compliance criteria were established through the undertaking of a detailed Receiving Water Assessment and consultation with the MOE. Appendix ‘C’ contains correspondence related to the effluent requirements. Table 6 summarizes the proposed effluent compliance criteria for the mechanical treatment plant.

Table 6: Effluent Design Criteria and Objectives

PARAMETER	COMPLIANCE PERIOD	CRITERIA
BOD_5 (mg/L)	Year-Round	25
TSS (mg/L)	Year-Round	25
Total Phosphorus (mg/L)	September 1 through April 30	0.3
Total Phosphorus (mg/L)	May 1 through August 31	0.2
Total Ammonia as N (mg/L)	September 1 to April 30	15
Total Ammonia as N (mg/L)	May 1 to May 31	15
Total Ammonia as N (mg/L)	June 1 to August 31	5
E. coli (org./100 mL)	Year-Round	–
Acute Lethality	Year-Round	Non-Acutely Lethal

2.3 Site Constraints

2.3.1 Geotechnical/Hydrogeological Considerations

Based on the Geotechnical/Hydrogeological Report prepared by Golder Associates Ltd. during Phase 2, the native subsoil in the area of the existing Lagoons consists of a surficial topsoil layer underlain by a stiff and weathered silty clay deposit that lies over bedrock (Golder, 2005). It was noted that the depth of the overburden material varies from about 3 to 8 metres in the vicinity of the existing Lagoon (Golder, 2005). Further geotechnical investigation will be required during a preliminary design stage after the completion of the Class EA.

The site was determined to be within the groundwater catchment area of a municipal water well. The potential impacts of temporary dewatering practices on the groundwater conditions will need to be considered and mitigated during detailed design and construction.

2.3.2 Natural Environmental Assessment

Based on existing drawings, discussions with the Town, visual examinations and previous Reports, land within the proposed project area has been significantly disturbed by previous construction activities. On this basis, no significant natural features are anticipated to be disturbed by the proposed works.

It is anticipated that the outlet structure will not require upgrades and the areas in and around the River will not be disturbed.

2.3.3 Archaeological Assessment

Based on existing drawings, discussions with the Town, visual examinations and previous Reports, land within the proposed project area has been significantly disturbed by previous construction and activities. On this basis, no significant areas of cultural, heritage or archaeological significance are anticipated to be disturbed by the proposed works.

2.3.4 Existing Facilities

The Town owns all of the land within and surrounding the proposed site for a mechanical treatment plant. Consideration must be made during detailed design and construction to ensure that the operation of the existing Lagoon is maintained until new facilities are constructed and commissioned.

2.4 Preferred Solution

2.4.1 Collection

During Phase 2, comments were received from the MOE requesting that the collection system also be considered within the scope of this Class EA. The collection system has been included in Phase 3.

2.4.2 Pumping

Similarly, the pumping stations also need to be considered with the scope of this Class EA. Pumping stations have also been included in Phase 3.

2.4.3 Treatment

During Phase 2, several treatment alternatives were identified and evaluated against a set of criteria in order to determine a preferred solution.

A Mechanical Treatment Plant with Tertiary Treatment was ultimately determined to be carried forward into Phase 3 as the preferred treatment solution. The concept to be evaluated in Phase 3 includes Raw Sewage Handling, Preliminary Treatment, Secondary (Biological) Treatment, Tertiary (Advanced) Treatment, Disinfection, and Biosolids Handling.

Figure 2 presents a conceptual site plan of a proposed Mechanical Treatment Plant located within the current site that was developed during Phase 2. At the conclusion of Phase 2, the project scheduling was reaffirmed as a Schedule 'C' Class EA undertaking.

3.0 REVIEW AND EVALUATION OF SEWAGE SYSTEM ALTERNATIVES

Phase 3 of the Class EA includes the evaluation of design alternatives for sewage collection, pumping and treatment concepts for the Almonte Ward Communal Sewage System. Figure 3 displays a simplified block flow diagram that outlines the various components of collection, pumping and the treatment process based on the preferred option selected in Phase 2.

A review of the design concepts has been undertaken for each of the following major system components:

- Raw Sewage Handling
- Preliminary Treatment
- Secondary (Biological) Treatment
- Tertiary (Advanced) Treatment
- Disinfection
- Biosolids Handling

In the following Sections of the Report, various “unit operations” have been evaluated under the above major categories.

4.0 RAW SEWAGE HANDLING

4.1 Collection System

The Almonte sewage collection system consists of approximately 35 km of vitrified clay, concrete pipe and Polyvinyl Chloride (PVC) gravity sewer pipes ranging in size from 150 mm to 600 mm in diameter. Figure 4 provides an overview of the collection system. Concrete storm sewers, ranging from 200 mm to 750 mm diameter, exist in newer developments and in areas with reconstructed streets (refer to Appendix ‘D’ for a database of the sewers).

Rain events can lead to excess flows at the existing sewage pumping stations. The Town recognizes that the reduction of infiltration and inflow into the sanitary collection system is important. In an effort to renew infrastructure and reduce flows, the Town continues to implement an ongoing multi-year program to inspect sewers, and prioritize infrastructure rehabilitation or replacement as required. Examples of sanitary sewers reconstructed to address aging infrastructure and reduce extraneous flows are listed in Appendix ‘D’.

The Town has also undertaken the following flow-reduction programs to optimize the existing service capacity and reduce future capacity requirements for the Almonte Ward:

- public information and educational programs outlining the advantages of water conservation;
- full water metering program in all locations has been in place for over 50 years; water meters and charges based on consumption encourage conservation;
- all new construction expected over the planning period now requires the use of more water efficient fixtures (e.g., low-flow toilets and shower heads); and
- sewers are video inspected every five years.

It is estimated that, currently, the average sewage flow (including extraneous flows) is approximately 540 L/capita/day. This is not considered significantly excessive as compared to typical MOE Guidelines (MOE, 1984) or communities of similar size. However, further renewing of portions of the collection system should continue on an annual basis and should be a key component in any preferred solution developed by this Class EA.

Annual investments may vary depending on Town budgets and infrastructure priorities. It would be beneficial for the Town to formalize a long term plan that incorporates renewal of all communal infrastructure.

4.2 Sewage Pumping

The Almonte Ward sewage collection system currently includes five small sub-area sewage pumping stations and one main sewage pumping station located on Almonte Street close to Gemmills Bay (refer to Figure 4). All sewage generated in the service area is ultimately conveyed to this Pumping Station. The Gemmills Bay Pumping Station houses two pumps in a dry well/wet well configuration.

The Gemmills Bay Pumping Station conveys wastewater via two forcemains (300 mm and 400 mm in diameter) to the Lagoon. The pumps formerly operated in a duty/standby arrangement when only one forcemain existed. Since the installation of a second forcemain in the 1990s, each pump is now dedicated to a respective forcemain.

In the area of the Gemmills Bay Pumping Station, there are two bypass channels to the Mississippi River that are used during high flow or emergency events. The first bypass point is located approximately 200 metres upstream of the Gemmills Bay Pumping Station. This bypass is alarmed and metered to allow Operations staff to record the time and volume of bypass events. There is no chlorine disinfection at this bypass, as the access point is a manhole located in the middle of Almonte Street. Chlorine pucks are used for disinfection, as required. The second bypass overflow is located in the wet well of the Pumping Station. Sodium hypochlorite equipment is available to automatically disinfect these bypass flows.

The Gemmills Bay Pumping Station is nearing the end of its originally intended service life and will require upgrading in the near future. There were 20 reported bypass events between 2002 and 2004, mostly the result of high flows. The total estimated quantity of all bypasses for this time period was reported to be 12,454 m^3, as compared to an estimated 2.7 million cubic metres of sewage treated at the Lagoon. This represents less than half a percent of the total sewage generated. Although the Station is capable of pumping the majority of the sewage, some increase to its capacity will be required over the next 20 years, along with additional upgrades and provisions for redundancy.

The impact of hydraulic requirements of the Mechanical Treatment Plant on the existing Pumping Station must also be considered. Raw sewage conveyed to the Gemmills Bay Pumping Station will need to be pumped up to the proposed treatment facility, allowing the wastewater to flow by gravity through the different treatment stages. As a result, upgrades to the existing Pumping Station may be required for these reasons as well.

Any design concepts for the Almonte Ward Communal Sewage System will incorporate upgrades to the Gemmills Bay Pumping Station to ensure that the facility is capable of meeting projected design flows from the community and the hydraulic requirements of a proposed Mechanical Treatment Plant.

The twin forcemain capacity appears to be sufficient to meet the 20-year planning period needs. The service life of the newer 400 mm diameter forcemain will meet the 20-year planning period, however, the condition of the older 300 mm diameter forcemain will need to be acceptable for the 20-year period. It would be prudent to budget for the replacement of this forcemain as part of this Class EA. Based on a unit replacement cost of $400/m, it is estimated that a new forcemain would be approximately $600,000 (excluding costs for contingencies and engineering).

It is also reasonable to assume that some small upgrades to the sub-area pumping station and forcemains may also be necessary. Annual condition assessments of these stations should be undertaken.

An opinion of probable costs to upgrade the Gemmills Bay Pumping Station is summarized in Table 7.

Table 7: Probable Costs for Pumping Station Upgrades

DESCRIPTION	COSTS
Site Works and Demolition/Removals	$100,000
Architectural/Structural	$300,000
Process Equipment and Piping	$700,000
Process Mechanical	$200,000
Odour Control Provisions	$100,000
Electrical and Instrumentation	$450,000
Forcemain	$600,000
Miscellaneous	$150,000
SUBTOTAL	**$2,600,000**
Contingency (20%)	$520,000
Engineering (15%)	$468,000
TOTAL	**$3,588,000**

4.3 Optional Septage Receiving Facility

As of 2001, approximately 60% of the residents in the amalgamated Town of Mississippi Mills used private septic systems and sewage holding tanks. The septage generated by these systems (collected by private haulers) is currently spread on fields, temporarily stored, or hauled to a sewage treatment facility. The Nutrient Management Act (2002) has specified that the practice of applying untreated septage on land is to discontinue by 2007. Suitable handling Options for septage are being reviewed by the MOE.

The potential for accepting septage at a new treatment facility is a consideration in this Class EA. It should, however, be noted that the scope of this Class EA does not include an exhaustive review of septage handling Options for the rural areas within the Town. Preliminary calculations indicated that co-treatment of the septage generated within the Town of Mississippi Mills could be incorporated as part of a new mechanical treatment plant.

Figure 5 illustrates the main components of a septage receiving facility. A modernized septage receiving facility at a treatment plant could include the following components:

- truck unloading area and septage receiving facility complete with an odour control system;
- automated septage screening, washing and grinding system;
- automated card metering system to identify haulers;
- automated sampler to monitor septage quality;
- flow monitoring and control; and
- equalization storage.

In addition to the costs associated with a septage receiving facility, calculations illustrate that organic loading of the main treatment process would significantly increase provided that the new plant were to handle all of the septage forecasted to be generated within the Town of Mississippi Mills over the 20-year planning period. This would result in somewhat larger process tankage and equipment, which, in turn, would result in a capital cost premium for the plant. It is difficult to determine a precise cost due to limited design detail at this stage, however, it would be reasonable to expect that the additional cost could equal the cost of the septage receiving facility. Therefore, a total cost premium of $1.5 million has been assumed if septage receiving at the scope indicated is incorporated into a new plant.

Table 8 summarizes the approximate costs associated with a septage receiving facility.

Table 8: Costs for Optional Septage Receiving Facility

DESCRIPTION	COSTS
Septage Receiving Building	$150,000
Septage Treatment Unit	$200,000
Concrete Equalization Tanks	$100,000
Process Mechanical	$50,000
Odour Control Provisions	$100,000
Electrical and Instrumentation	$150,000
Cost Premium for Plant	$750,000
SUBTOTAL	**$1,500,000**
Contingency (20%)	$300,000
Engineering (15%)	$270,000
TOTAL	**$2,070,000**

Another alternative is not to include any provision for septage handling within the scope of the new works. It is possible that other less expensive or more attractive options will be available to septage haulers (e.g., lime stabilization).

A third option would be to size the main tanks to possibly accept additional organic loading in the future, but to not include septage receiving at this time (i.e., include it only as a possible future upgrade). In this case, capital costs would be about half of those presented in Table 7.

5.0 PRELIMINARY TREATMENT

Wastewater contains large solids and grit that can interfere with treatment processes or cause undue mechanical wear and increased maintenance on wastewater treatment equipment. To minimize potential problems, these materials require separate handling. Preliminary treatment considered in this Class EA include screening, grit removal and flow equalization.

5.1 Screening

Screening is typically the first unit operation used at sewage treatment plants. Screening removes objects such as rags, paper, plastics and metal to prevent damage and clogging of downstream equipment, piping, and appurtenances. For the Almonte Sewage Treatment Plant (STP), there are no significant choices of screening technology and only bar screening will be considered for this stage of treatment. This screening technology makes use of bar screens to remove coarse solids.

Bar screens may either be cleaned manually or automatically. Although manual cleaning results in low capital costs, it is typically not employed as a dedicated screening solution due to the risks associated with operational hazards (i.e., odour, hygiene, etc.) and plant flooding or bypassing due to screen blockage.

Automatically cleaned or mechanical bar screens are a common form of wastewater screening technology at many plants. Automated cleaning of the bar screens is achieved using an escalator type system to continuously clean the screens based on a timed cycle or differential head of the upstream and downstream channels. With improved screenings, washing and compaction systems, screens with smaller openings are becoming more common and are now installed in several plants in Ontario. The main advantages of smaller openings are the return of organics into the wastewater stream and reduction of inorganic screening material passing through to the downstream processes. Although initially more costly than the manually cleaned alternative, it provides a reliable method of screening removal that is compatible with unattended plant operations.

Costs associated with the Screening Removal Stage are outlined in Table 9.

Table 9: Costs for Screening Removal Stage

DESCRIPTION	COSTS
Screening Room[1]	$100,000
Concrete Channels	$100,000
Screening Equipment and Bypass Screen	$150,000
Process Mechanical	$50,000
Odour Control Provisions[1]	$50,000
Electrical and Instrumentation[1]	$50,000
SUBTOTAL	**$400,000**
Contingency (20%)	$80,000
Engineering (15%)	$70,000
TOTAL	**$550,000**

1 Screening and Grit Removal Equipment will likely be housed in the same building. Costs associated with Screening Room, Odour Control and Electrical are shared with Grit Removal design.

5.2 Grit Removal

Grit includes sand, gravel, cinder or other solids other than the organic biodegradable solids in the wastewater. Grit removal is provided to protect mechanical equipment from unnecessary abrasion and wear, and prevent grit deposition in downstream channels and treatment tanks.

Aerated and vortex grit removal are common grit removal technologies and are considered suitable for most secondary treatment technologies. Horizontal flow grit chambers (i.e., detritors) equipped with mechanical collectors similar to a sedimentation tank are noted as an option but are not common. It is understood that horizontal flow grit chambers also remove a high percentage of settleable organic matter that is not beneficial to the treatment process (JLR, 2004).

5.2.1 Aerated Grit Removal

Aerated grit removal is accomplished by causing wastewater to flow in a spiral pattern by introducing air into a tank along one side resulting in a perpendicular spiral velocity pattern in the chamber. Heavier particles accelerate and diverge from the streamlines, dropping to the bottom of the tank, while lighter organic particles are kept in suspension and eventually carried out of the tank to downstream processes.

The main components in an aerated grit removal system include:

- grit chambers
- blower and air distribution system
- screw collector and grit pumping
- grit classifier and disposal.

5.2.2 Vortex Grit Removal

Vortex-type grit removal generally consists of a cylindrical tank that creates a vortex-type flow pattern. Grit settles by gravity to the bottom of the tank into a grit hopper, while effluent exits at the top of the tank. Tanks may be constructed using a concrete vessel or a manufactured steel tank. As these systems are not aerated, odour generation is reduced, particularly if grit is removed from the bottom of the hopper in an enclosed system, such as pumping. A common type of vortex grit removal system utilizes rotating paddles to maintain minimum flow velocities necessary to suspend organic matter. Similar to the aerated grit removal alternative, grit settles to the bottom of the tank and is removed by pumping the slurry to a grit classifier, which separates grit from the wastewater.

Main components in an aerated grit removal system include:

- vortex tanks
- paddles suspension system
- grit hopper and grit pumping
- grit classifier and disposal.

5.2.3 Summary of Costs for Grit Removal Alternatives

Table 10 provides a summary of comparative capital costs for both technologies.

Table 10: Summary of Grit Removal System Costs

DESCRIPTION	Aerated Grit Chamber	Vortex Grit Chamber
Grit Handling Room[1]	$50,000	$80,000
Concrete Tank	$100,000	$100,000
Grit Removal and Pumping Equipment	$150,000	$250,000
Process Mechanical	$50,000	$20,000
Odour Control Provisions[1]	$50,000	$50,000
Electrical and Instrumentation[1]	$50,000	$50,000
SUBTOTAL	**$450,000**	**$550,000**
Contingency (20%)	$90,000	$110,000
Engineering (15%)	$81,000	$100,000
TOTAL	**$621,000**	**$760,000**

1 Screening and Grit Removal Equipment will likely be housed in the same building. Costs associated with Screening Room, Odour Control and Electrical are shared with Grit Removal design.

Aerated grit removal requires more energy and generates more odours than vortex grit removal due to the requirements for continuous aeration. An air requirement of approximately 125 L/s (based on typical air supply rates of 0.5 m^3/min/m and tank lengths of approximately 7 m) is estimated to cost between $10,000 and $12,500/year (based on $0.08 to $0.10/kWh), assuming blowers are operated 24 hours per day, and seven days a week (Metcalf & Eddy, 2003). The annual estimated energy costs for vortex grit tanks are between $2,000 and $3,000 (based on $0.08 to $0.10/kWh), assuming equipment is operated 24 hours per day, seven days per week. The vortex system also requires less maintenance due to having fewer moving parts, so the anticipated operations and maintenance costs associated with this system would be lower than the aerated grit system.

5.2.4 Evaluation of Grit Removal Alternatives

Table 11 compares the advantages and limitations of the grit removal technologies outlined above.

Table 11: Advantages and Limitations of Grit Removal Alternatives

TECHNOLOGY	ADVANTAGES	LIMITATIONS
Aerated Grit Removal	• Consistent removal efficiency over a wide flow range • Proven technology for plants of similar scale • Ability to adjust performance and condition within tank • Performance of downstream units may be improved using pre-aeration to reduce septic conditions in incoming wastewater • Aerated grit chambers are versatile allowing for chemical addition, mixing, pre-aeration, and flocculation (EPA, No Date) • Lower capital costs • Aerated grit chambers generally use non-proprietary equipment providing flexibility with process adjustments and maintenance	• Larger footprint • Odour impacts associated with air addition • Mechanically more complex than vortex grit removal system • Higher operations and maintenance costs
Vortex Grit Removal	• High percentage removal of fine grit • Consistent removal efficiency over a wide flow range • Minimal head loss through the system (EPA, No Date) • Proven technology for plants of similar scale • Lower odour control impacts • Smaller footprint • Energy efficient and few mechanical parts resulting in lower operations and maintenance costs	• Minimal grit storage • Vortex equipment housed indoors and will require a larger building footprint • Vortex units are usually deep and may require special design and construction considerations resulting in increased capital costs • Higher capital costs • Vortex systems are usually of proprietary design making modifications more difficult • Paddles tend to collect rags (EPA, No Date)

Based on the advantages and limitations presented in Table 11, both aerated and vortex grit removal are considered feasible options for a new plant. Aerated grit removal is capable of providing consistent removal efficiency over a wide flow range and has lower initial capital costs. Another consideration of the aerated grit removal option is that it is a proven technology that may improve downstream treatment through pre-aeration of the raw sewage. Vortex grit removal has a lower estimated life-cycle and would result in lower maintenance costs, as well as fewer health and safety concerns related to odours.

Therefore, either of these two alternatives would be appropriate and should be carried forward for further consideration during the preliminary design phase of the project.

5.3 Optional Equalization Storage

Flow equalization is a method used to overcome operational problems associated with variation of raw sewage flow rates and characteristics at wastewater facilities. The use of equalization storage improves downstream processes and, at times, can reduce the size and cost of downstream processes (Metcalf & Eddy, 2003).

Flow equalization is the dampening of flow rate variations to achieve constant or nearly constant flow rates. For the purpose of this Class EA, the principal application of the storage would be to equalize wet weather flows (i.e., divert flows greater than the peak instantaneous flow of 18,000 m^3/day) to minimize shock loading, improve biological processes by ensuring a consistent solids loading, and assist with chemical feed control and reliability. It should be noted that flow equalization is generally needed with some secondary treatment technologies (e.g., Membrane Bioreactors) and will be discussed in later sections of this Report.

Typically, equalization basins can be constructed of earthen, concrete or steel materials (Metcalf & Eddy, 2003). For the concept presented in Phase 2 of the Class EA, it was proposed that a portion of the existing Lagoon be retrofitted to provide equalization storage. This could provide relatively low cost equalization storage.

In Phase 2, it was assumed Cell C would be used for equalization storage, however, upon further review, it may be beneficial to utilize Cell ‘AA’ (out of service) since there would be little, if any, sludge accumulated in this cell. The volume of equalization storage available in Lagoon Cell ‘AA’ is summarized in Table 12.

Table 12: Optional Volumes of Equalization Storage

USED AREA OF CELL 'AA'	AVERAGE LIQUID DEPTH (m)	VOLUME [1] (m^3)
25%	1.82	2,730
50%	1.82	5,460
75%	1.82	8,190
100%	1.82	6,000

[1] Based on a cell area of 6000 m^2 and a depth of 2.66 m.

Costs associated with the Optional Equalization Storage are provided in Table 13.

Table 13: Costs for Optional Equalization Storage

DESCRIPTION	COSTS
Earthworks and Piping	$50,000
Pumping Station	$50,000
Pumping Equipment	$50,000
Process Mechanical	$30,000
Electrical and Instrumentation	$20,000
SUBTOTAL	**$200,000**
Contingency (20%)	$40,000
Engineering (15%)	$36,000
TOTAL	**$276,000**

5.4 Summary of Preliminary Treatment

The preliminary treatment system for the Almonte Ward Communal Sewage System will consist of the following components:

- Bar Screening

- Aerated or Vortex Grit Removal
- Optional Equalization Storage.

6.0 SECONDARY (BIOLOGICAL) TREATMENT

The purpose of secondary (biological) treatment of domestic wastewater is to:

- transform dissolved and biodegradable constituents into acceptable end products;

- capture and incorporate suspended and non-settleable solids into a floc;

- transform or remove nutrients in the wastewater, such as nitrogen and phosphorous; and,

- in some cases, remove specific trace organic constituents and reduce the concentration of organic and inorganic compounds (Metcalf & Eddy, 2003).

Secondary treatment options for the Almonte STP are wide-ranging and may include Activated Sludge Processes (ASP) (i.e., Conventional, High-Rate, Extended Aeration, Sequencing Batch Reactor), Rotating Biological Contactor (RBC), Moving Bed Biofilm Reactor (MBBR), Sequencing Batch Reactor (SBR), Membrane Bioreactor (MBR), and Biological Aerated Filter (BAF).

Technologies that were excluded from the comparison include the Upflow Sludge Blanket Reactor (USBR). Although successful at treating municipal wastewater, the USBR technology was not considered in this comparison because it is our understanding that there are currently no applications of this technology for facilities greater than 1,000 m^3/day operating in North America.

Common variations of the Activated Sludge Process (ASP) are presented within the following sections of the Report, with the exception of Biological Nutrient Removal (BNR). Generally, the operations associated with a BNR plant are more complex than conventional or other common ASP processes. Although a viable option for wastewater treatment in certain cases, it is also assumed that this technology may not be practical for consideration due to level of care and management of environmental conditions required to meet the required performance criteria (i.e., phosphorus removal and nitrification).

6.1 Traditional Activated Sludge Processes (ASPs)

The Activated Sludge Process (ASP) is one of the most widely used secondary treatment technologies. There are many variations of the Activated Sludge Process but all essentially consist of an aerated biological reactor followed by a secondary clarifier. In the biological reactor, commonly known as an aeration tank, suspended biomass degrades the organic material and subsequently separates it from the wastewater in the secondary clarifier. Thickened biomass from the clarifier underflow is recycled to the aeration tank to maintain a desired biomass concentration.

The ASP is a robust and well-proven process for treating wastewater under widely varying environmental conditions due to its operational flexibility. The process allows operators to optimize the process for a given set of environmental conditions by adjusting the biomass inventory and sludge handling.

Biological ammonia oxidation, a process called nitrification, in which ammonia is converted to nitrate during the secondary treatment stage, can be accomplished by operating the process at the required solids retention time while supplying adequate oxygen. Phosphorus removal can be accomplished through chemical addition before, during, or immediately following the aeration tank.

With the implementation of an ASP, it would be necessary to provide sludge handling facilities. For example, it may be necessary to provide co-thickening of the Waste Activated Sludge (WAS) in the primary clarifiers, or to thicken the WAS separately (JLR, 2004).

6.1.1 Conventional Activated Sludge (CAS)

The Conventional Activated Sludge (CAS) process typically consists of long rectangular aeration basins where primary effluent is blended with Return Activated Sludge (RAS). Aeration is typically supplied through air diffusers or mechanical aerators. CAS systems employ primary clarification upstream of the aeration basins.

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 system typically removes 90 to 95 percent of the raw sewage BOD_5, and could be designed to nitrify year-round, based on adequate oxygen transfer, aeration tank and secondary clarifier capacities. MOE Guidelines identify that effluent quality from a CAS system is typically 15 mg/L for BOD_5 and 15 mg/L for TSS. The effluent ammonia concentration in a conventional activated sludge treatment plant with nitrification typically ranges from 1 to 5 mg/L (MOE, 1984). With chemical addition, total phosphorus effluent levels less than 1 mg/L are readily achieved with the CAS process.

Table 14 presents estimated footprint requirements for a CAS plant for the Almonte STP, which are based on the MOE Guidelines.

Table 14: Estimated CAS Plant Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	180 m^2 (1)
Bioreactor (Aeration)	250 m^2 (4.5 m tank depth)(2)
Secondary Clarification	300 m^2 (3)

1 Based on Surface Settling Rate of 50 m/day at peak flows (MOE, 1984)
2 Based on Hydraulic Design Criteria of 6 hour retention time and 4.5 m tank depth (MOE, 1984)
3 Based on Surface Settling Rate of 30 m/day at peak flows (MOE, 1984)

Step-Feed ASP is a modification of the CAS process that provides the ability to feed wastewater as it enters at two or more points along the length of the aeration tank. This type of treatment avoids solids washout during storm flow events by feeding the wastewater towards the end of the aeration basin and retaining the solids in the front portion of the tank. This process allows the operator to maintain constant oxygen requirements along the length of the tank and decreases the effect of peak hydraulic and organic loads (JLR, 2004). One disadvantage of this type of system is BOD removal rates and total phosphorous removal rates, and nitrification is generally lower during step-feed operations, due to shorter contact times in the aeration basin.

If CAS is selected as the preferred secondary treatment technology for the Almonte STP, capability to operate the process in a step-feed mode should be considered in the design phase.

6.1.2 Extended Aeration (EA)

The Extended Aeration (EA) process is a variation of the Conventional Activated Sludge process in which there is no primary clarification. The EA process uses longer retention time (approximately 15 hours) in the aeration stage compared to the CAS process. The longer Hydraulic Retention Time (HRT) provides greater process resilience to shock loading and ability to treat high flows.

This process is preferred for the treatment of higher strength wastewater and is typically implemented following preliminary treatment (i.e., screening and grit removal) (JLR, 2004). MOE Guidelines identify that EA systems generally provide the same effluent quality as a CAS process. EA systems typically achieve 15 mg/L for BOD_5 and 15 mg/L for TSS effluent quality. The effluent ammonia concentration in an extended aeration treatment plant typically ranges from 1 to 5 mg/L (MOE, 1984). With chemical addition, total phosphorus effluent levels less than 1 mg/L are readily achieved with the EA process.

Table 15 presents conceptual design requirements for an EA plant for the Almonte STP Due to longer HRT, the EA process typically has larger aeration tank volumes than those used for CAS systems. The secondary clarification capacity for the EA process would be similar to a CAS plant.

Table 15: Estimated EA Plant Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	Not Required
Bioreactor (Aeration)	625 m^2 (4.5 m tank depth)(1)
Secondary Clarification	300 m^2 (2)

1 Based on Hydraulic Design Criteria of 15 hour retention time and 4.5 m tank depth (MOE, 1984)

2 Based on Surface Settling Rate of 30 m/day at peak flows (MOE, 1984)

An oxidation ditch is a modified EA system that also utilizes long retention times ranging from 6 to 30 hours (EPA, 2000). Typical oxidation ditch treatment systems consist of a single or multi-channel configuration within a ring, oval or horseshoe-shaped basin. Due to the ‘shallow racetrack’ type reactors, this technology requires more land than CAS systems. Similar to the EA process, flow to the oxidation ditch is aerated and mixed with return sludge from a secondary clarifier. Preliminary treatment precedes the oxidation ditch and primary settling is typically not required.

The oxidation ditch process is a fully demonstrated secondary wastewater treatment technology, applicable in any situation where EA or CAS treatment technologies are being considered (EPA, 2000). Oxidation ditches are applicable in plants that require nitrification because the basins can be sized using an appropriate solids retention time to achieve nitrification at the mixed liquor minimum temperature (EPA, 2000). In the case that the EA process is selected as the preferred secondary treatment technology, the oxidation ditch could be carried forward for review during the preliminary design phase.

6.1.3 High-Rate Activated Sludge Process (High-Rate ASP)

The High-Rate ASP uses a completely mixed aeration tank operated at a short HRT of approximately 4 hours, high sludge recycle and high organic loading rate. Adequate mixing is very important in order for this process to run effectively.

It is understood that High-Rate ASP systems can achieve 75 to 90 percent BOD removal, but more operational attention is required as these processes are sensitive to high return sludge rates that result in high solids loading rates to the secondary clarifiers (JLR, 2004). This process is especially prone to effluent suspended solids exceedences during wet weather flow periods, due to the high operating biomass concentrations and the small bioreactors. As such, this process has less operational flexibility than a CAS or EA system, and the effluent BOD and TSS levels from the High-Rate ASP are typically slightly higher than the aforementioned systems. Year-round nitrification performance of High-Rate ASP can be achieved provided that the secondary clarifiers are designed to handle higher solids loading rates during cold weather periods (JLR, 2004). With chemical addition, total phosphorous effluent levels similar to the CAS and EA systems are attainable.

This process would offer a one-third reduction in the surface area requirements of the aeration tanks, compared to the CAS process. Due to higher mixed liquor concentrations, the secondary clarifier surface area requirements for this process would be two to four times greater than those for the CAS process. It should be noted that primary clarification will be required for this type of system.

Table 16 provides a summary of the footprint requirements for a High-Rate ASP system for the Almonte STP.

Table 16: Estimated High-Rate Activated Sludge Plant Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	180 m^2 (1)
Bioreactor (Aeration)	170 m^2 (4.5 m tank depth)[2]
Secondary Clarification	600 m^2 (3)

1 Based on Surface Settling Rate of 50 m/day at peak flows (MOE, 1984)
2 Based on Hydraulic Design Criteria of 4 hour retention time and 4.5 m tank depth (MOE, 1984)
3 Estimated value based on past case studies at peak flows (JLR, 2004)

6.1.4 Sequencing Batch Reactor (SBR)

The Sequencing Batch Reactor (SBR) process is a variation of the ASP that uses a “fill and draw” mode of treatment that is capable of providing high quality effluent. Unlike the CAS and EA systems, which provide continuous flow through a series of tanks, the SBR is a time-based, batch system that can satisfy different treatment objectives by modifying the application and duration of mixing, aeration and settling all within a single tank, making the SBR process very flexible.

A typical operating sequence for SBR is composed of the following five stages: fill, react (aeration), settle (clarification), draw and idle. Sludge wasting is generally conducted during the settle or idle stages, but can occur in the other phases, depending on the mode of operation. 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.

One of the disadvantages of an SBR is that the discharged flow from the basins tends to be much higher than a “continuous flow-through” process. This requires that, either downstream processes (e.g., disinfection, filters, etc.) be sized larger, or that some form of equalization be provided, adding capital costs to the works required.

The BOD_5, TSS and ammonia removal performance efficiency of the SBR process is expected to be similar to that achieved in a CAS or EA process. Chemical phosphorous removal can be achieved in an SBR by adding chemical to the SBR feed. Alternatively, phosphorous removal can be achieved using specially designed SBRs equipped with Biological Nutrient Removal (BNR) capabilities by producing the cyclically alternating aerobic, anoxic and anaerobic conditions required by the BNR process (JLR, 2004). An SBR designed for biological phosphorus removal can achieve approximately 1 mg/L total phosphorous in the effluent.

Based on past experience, average design for SBR systems range from 12 hours, where the objective is to meet organic and suspended solids reduction, to 24 hours, where flow rates are highly variable and nitrification, denitrification and /or biological phosphorous removal is also required. Primary and secondary clarification tanks are not required. Table 17 presents footprint requirements for a possible SBR system for the Almonte STP, which were estimated based on supplier information (Envirocan, 2006).

Table 17: Estimated SBR Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	Not Required
Bioreactor (SBR)	2 Reactors: 1,000 m^2 [(1)]
Secondary Clarification	Not Required

[1] Based on two SBR reaction tanks as per supplier information (Envirocan, 2006)

Many relatively small SBR installations exist throughout Ontario including, but not limited to Cardinal Wastewater Treatment Plant, Petawawa Water Pollution Control Plant, Pembroke Water Pollution Control Plant, Long Sault Water Pollution Control Plant and Rockland Water Pollution Control Plant.

6.2 Attached Growth Processes

6.2.1 Rotating Biological Contactor (RBC)

The Rotating Biological Contactor (RBC) process typically consists of a reactor and a secondary clarifier. Typically, RBCs are preceded by preliminary clarification.

The RBC is an attached growth process where biomass grows on the surface of a rotating disc kept in partial submergence (i.e., approximately 40%). The RBC process typically involves a flow-through arrangement comprised of a number of passes of rotating biological contactors in series. RBCs are usually indoors or covered with fiberglass enclosures in colder climates, to minimize heat loss. The rotation of the media carries a film of wastewater that contacts with air, supplying oxygen for biological growth and oxidation on the media surface. As the thickness of the biomass 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 not an effective way to control operation and performance (i.e., final effluent quality), which, in an ASP, is accomplished by regulating the biomass inventory (JLR, 2004). Thus, RBCs provide fewer opportunities for treatment optimization or fine-tuning of the process and, as a result, can be subject to effluent quality variations. A benefit of RBCs is their low energy usage as they do not rely on blower energy for aeration.

One of the most important design considerations for RBCs 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 (JLR, 2004). Odour problems have also been reported with overloaded first stage processes. Recent design modifications have mitigated some of these issues. For example, step feeding and air driven RBCs are less prone to biomass buildup.

Effluent quality from an RBC is typically 7-15 mg/L for BOD_5 and less than 2 mg/L for ammonia can be achieved in a combined BOD removal and nitrification process (Metcalf & Eddy, 2003). The biomass from a RBC process can have poor settling characteristics that may, in turn, lead to a larger secondary clarifier surface area compared to an ASP facility.

Table 18 presents footprint requirements for a RBC for the Almonte STP, which were estimated based on supplier information (John Meunier, 2006).

Table 18: Estimated RBC Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	180 m^2 (1)
Bioreactor (RBC)	1800 m^2 (2)
Secondary Clarification	300 m^2 (3)

1 Based on Surface Settling Rate of 50 m/day at peak flows (MOE, 1984)
2 Based on supplier information (John Meunier, 2006)
3 Estimated value based on past case studies (JLR, 2004)

Some RBC installations in Ontario include the Guelph and the Niagara Falls Stamford Water Pollution Control Plants (each approximately 50,000 m^3/day), as well as Vankleek Hill and Wendover (smaller applications).

6.2.2 Moving Bed Biofilm Reactor (MBBR)

The hybrid Moving Bed Biofilm Reactor (MBBR) process is an example of the integrated fixed film activated sludge process. The process is based on the biofilm principle and is essentially a hybrid between a suspended growth and a fixed film system. The MBBR process consists of an aeration basin filled with suspended media and a secondary clarifier. This technology is a proprietary based system with a number of media suppliers.

The core of the process is the suspended media, designed to provide a large surface area for biofilm growth, which enables considerably higher biomass inventories to be carried in a given tankage volume compared to ASP processes (JLR, 2004). This can result in a reduction in required footprint of the secondary treatment facility relative to traditional ASP processes. The biofilm carrier elements are kept in suspension by air addition in aerobic reactors and by mixers in anoxic and/or anaerobic reactors. Barriers are provided within the biological reactors to retain the biofilm carriers within the tank.

Similar to the RBC, the MBBR system provides little control over the mass of organisms in the reactor, thus there is no effective way of controlling effluent quality under varying conditions. The BOD_5 and TSS removal efficiencies of the suspended/attached growth process would be similar to those achieved at a CAS or EA facility (JLR, 2004). Chemicals could be added before the secondary clarifier for precipitation of phosphorus and the process can be configured to meet strict nitrification requirements by adding additional media to the reactor or providing larger reactors.

This technology can be used to increase the capacity of existing facilities and may be better suited to retrofits.

Table 19 presents footprint requirements for an MBBR process for the Almonte STP, which was based on information provided by a media supplier (H2Flow, 2006). This technology does require the implementation of primary clarification.

Table 19: Estimated MBBR Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Primary Clarification	180 m^2 (1)
Bioreactor (MBBR)	1000 m^2 (2)
Secondary Clarification	300 m^2 (3)

1 Based on Surface Settling Rate of 50 m/day at peak instantaneous flows (MOE, 1984)
2 Based on supplier information (H2Flow, 2006)
3 Estimated value based on past case studies (JLR, 2004)

There are no examples of full-scale operating facilities in Ontario, although testing and piloting is being undertaken by the City of Toronto (Lakeview Wastewater Treatment Plant and Humber Wastewater Treatment Plant) and City of Hamilton (Waterdown Wastewater Treatment Plant) (JLR, 2004).

6.3 Advanced Activated Sludge Processes

6.3.1 Membrane Bioreactor (MBR)

The Membrane Bioreactor (MBR) technology represents an advanced activated sludge wastewater treatment process that is capable of producing very low suspended solids, tertiary level treated quality effluent. The process combines a biological reactor with a membrane filtration system for retention of the activated sludge.

The most common MBR process configuration for the treatment of wastewater involves submerging the membranes in the aeration tanks providing direct contact with the mixed liquor, essentially eliminating the need for external clarification and tertiary treatment. Because the need for sedimentation is eliminated and the biological process can operate at much higher mixed liquor concentration, the process tankage is significantly reduced. A dedicated aeration system below the immersed membrane module is used for membrane scouring and control of fouling and flux. Maintenance of an acceptable filtrate flux requires relatively high-energy input and cleaning of the membranes with chemicals. This involves regular in-place mechanical and chemical cleaning and, occasionally, more extensive chemical cleaning of the membranes in an external cleaning tank.

A typical MBR process achieves effluent BOD_5 and TSS concentrations of 2 mg/L or less, respectively. Ammonia concentrations of 2 mg/L can be attained during winter conditions (JLR, 2004). MBRs are capable of providing a high level of phosphorous removal and typically do not require an additional tertiary treatment stage.

Footprint requirements for a possible MBR system for the Almonte STP will be relatively small compared to most options. For optimal membrane performance, effective fine screening and grit removal is required. Provisions for building enclosures and equalization storage must also be considered. As previously noted, MBR technology does not require tertiary treatment.

The MBR technology is relatively new and, as such, is considered to be less proven and operationally more complex than CAS or EA systems. A full-scale demonstration of an MBR system was undertaken in Milton, Ontario, for the treatment of primary treated effluent (JLR, 2004). There are some small MBRs in operation in Canada. One of the largest MBR facilities in Canada was commissioned in Port McNicoll (design average day flow of 1,400 m^3/day) (Zenon, No Date).

6.3.2 Biological Aerated Filters (BAF)

Biological Aerated Filters (BAF) are high-rate biological reactors that utilize the features of an attached growth biological process and the efficient oxygen transfer capabilities of diffused aeration systems. The BAF process consists of a biological reactor that directs wastewater upward through a granular media bed, co-current with the air supply. The influent solids and biomass produced in the BAF accumulate in the filters and are removed by periodic backwashing.

The BAF technology is compact, owing to concentrated biomass and the combined function of biological treatment and solids separation within a single reactor. The BAF treatment technology is typically used to treat weaker wastewaters that have low organic and solids loads, and is, therefore, highly dependent on primary treatment effluent quality. As such, primary clarification always precedes BAF reactors.

Effluent with TSS and BOD_5 concentrations of less than 10 mg/L can be consistently produced. Effluent ammonia concentrations of less than 1.0 mg/L can be achieved in a combined BOD removal and nitrification process. Chemicals for phosphorous removal should be added to the primary treatment stage. Coagulant could be added immediately upstream of the BAF units, however, this practice may plug the filters and reduce the filter run time. Based on experience, to achieve phosphorus concentrations of less than 0.2 mg/L, tertiary treatment may also be required.

Generally, BAF plants have one of the smallest tankage requirements of all options. A major portion of the plant tankage consists of the primary clarifiers, backwash supply and backwash wastewater. The BAF treatment technology presents an opportunity to eliminate the need for tertiary treatment required to meet the stipulated phosphorus removal levels.

6.4 Summary of Costs for Secondary Treatment Alternatives

Table 20 summarizes the capital costs for the Secondary Treatment Alternatives.

Table 20: Cost Summary for Secondary Treatment Alternatives

DESCRIPTION	CAS	EA	HIGH-RATE AS	SBR	RBC	MBBR	MBR	BAF
Site Works	$500,000	$450,000	$500,000	$450,000	$500,000	$450,000	$450,000	$500,000
Flow Equalization Tanks	N/R	N/R	N/R	N/R	N/R	N/R	$1,000,000	N/R
Primary Treatment Tanks	$1,000,000	N/R	$1,000,000	N/R	$1,000,000	$1,000,000	N/R	$1,000,000
Biological Reactor Equipment	$100,000	$150,000	$150,000	$750,000	$750,000	$1,550,00	$3,900,000	$1,500,000
Secondary Treatment Tanks	$575,000	$725,000	$1,250,000	$1,750,000 (2)	$2,000,000	$1,250,000	$500,000	$500,000
Secondary Clarification Tanks	$1,100,000	$1,100,000	$1,100,000	N/R	$1,100,000	$1,100,000	N/R	N/R
Secondary Treatment Building (Blowers and RAS Handling)	$375,000	$375,000	$375,000	$350,000	$475,000	$375,000	$375,000	$375,000
Process Mechanical	$600,000	$500,000	$600,000	$500,000	$475,000	$600,000	$600,000	$600,000
Odour Control Provisions	$100,000	$100,000	$100,000	$100,000	$100,000	$100,000	$100,000	$100,000
Electrical and Instrumentation	$600,000	$600,000	$600,000	$600,000	$600,000	$600,000	$600,000	$600,000
SUBTOTAL	**$4,950,000**	**$4,000,000**	**$5,675,000**	**$4,500,000**	**$7,000,000**	**$7,025,000**	**$7,525,000**	**$5,175,000**
Contingency (20%)	$990,000	$800,000	$1,135,000	$900,000	$1,400,000	$1,405,000	$1,505,000	$1,035,000
Engineering (15%)	$891,000	$720,000	$1,021,500	$810,000	$1,260,000	$1,264,500	$1,354,500	$931,500
TOTAL	**$6,831,000**	**$5,520,000**	**$7,831,500**	**$6,210,000**	**$9,660,000**	**$9,694,500**	**$10,384,500**	**$7,141,500**

1 N/R: Not Required
2 Cost includes Equalization Storage Tanks and Pumping

6.5 Evaluation of Secondary Treatment Alternatives

Eight Secondary Treatment Alternatives have been evaluated based on technical requirements. Table 21 summarizes the advantages and limitations of each alternative.

Table 21: Advantages and Limitations of Secondary Treatment Alternatives

ALTERNATIVE	ADVANTAGES	LIMITATIONS
CAS	• Proven and effective secondary treatment process for similar scale and climates • Less operating costs than other traditional ASPs due to smaller aeration tank requirements • Continuous flow reactor capable of handling flow variations • Provides operators with the ability to optimize process through oxygen transfer (fine pore air diffusers) and sludge management • Design parameters are clearly established • Potential for expandability • Can be modified to step-feed process	• CAS systems generally produce a dense primary sludge that requires anaerobic digestion. • Sludge settleability can be a concern • Higher capital costs attributed to required infrastructure (i.e., primary and secondary clarifiers) • Moderately large footprint
EA	• Proven and effective secondary treatment process for similar scale and climates • Simple operating parameters • Provides nitrification due to higher sludge retention times • Continuous flow reactor capable of handling flow variations • Provides operators with the ability to optimize process through oxygen transfer (fine pore air diffusers) and sludge management • Design parameters are clearly established • Reduction in footprint (compared to CAS) • Potential for expandability	• Higher operating costs associated with air requirements • Moderate capital costs attributed to larger aeration tanks and secondary clarifiers

High-Rate AS	• Offers similar advantages as the CAS and EA. Year-round nitrification performance of High-Rate ASP can be achieved provided that the secondary clarifiers are designed to handle higher solids loading rates during cold weather periods • Slightly smaller aeration tank requirements than CAS • Potential for expandability	• Less operational flexibility than a CAS or EA system and the effluent BOD and TSS levels from the High-Rate ASP are typically slightly higher • Operational attention is required as these precesses are sensitive to high return sludge rates that result in high solids loading rates to the secondary clarifiers
SBR	• Proven and effective secondary treatment process for similar scale and climates • Eliminates need for primary and secondary clarifiers • Reduction in footprint (compared to CAS and EA) • Easily modified to a BNR process • Large flow variations can be effectively treated • Provides operators with the ability to optimize process through oxygen transfer (fine pore air diffusers) and sludge management • No return sludge pumping associated with SBR, reducing operational costs as compared to CAS and EA • Potential for expandability	• Higher discontinuous discharge flows from SBRs may require equalization storage and pumping or larger sizing of equipment downstream of the reactors • Process relies on a sophisticated control and timing system • SBR systems generally produce a sludge that requires aerobic digestion
RBC	• Low energy requirements • Continuous flow reactor capable of handling flow variations • Smaller footprint than ASP plants • Potential for expandability • Simple process • Modular process providing upgrading potential	• Process relies on a fixed growth process and does not provide operators with the ability to optimize or control effluent characteristics • Moderate capital costs attributed to RBC tanks and enclosures • Unreliable operation due to history of mechanical failures caused by high loading in the first stage of treatment • No recent installation of the RBCs in Ontario

MBBR	• Provides effective secondary treatment due to higher biomass inventory • No return or backwash pumping requirements • No primary reactors required	• Fine screening required upstream • Less common technology • Higher operating costs associated with air requirements due to reduced oxygen transfer and higher dissolved oxygen set-point (coarse air diffusers) • Provides limited control over the mass of organisms in the reactor reducing control of effluent quality under varying conditions
MBR	• Enhanced quality effluent • Small footprint requirements	• High energy and maintenance requirements • High capital costs and operational complexity • Limited installations of this scale and climate • Although providing enhanced quality effluent, cold weather applications may require careful consideration with respect to nitrification • Relatively new technology and, therefore, there is a relative uncertainty with respect to membrane life

BAF	• Enhanced quality effluent • Small footprint requirements • Provides the ability for aeration cost savings during low flow periods • No secondary clarifiers required • Modular process providing potential for upgrades	• High capital costs and operational complexity • Requires backwashing. Backwash water and associated dilute waste solids produced that would require additional primary capacity for sludge thickening • Although providing enhanced quality effluent, the process still may require tertiary treatment • Limited installations of this scale and climate • Potential loss of media • Requires intermediate pumping between the primary treatment stage and BAF reactors • Sophisticated preliminary and primary treatment required • Phosphorous removal achieved in primary clarifiers

6.6 Secondary Treatment Summary

The review of the preferred secondary treatment technology for the Almonte STP needs to reflect and evaluate priorities that are unique to the community. Key considerations identified for the review and general comparison include:

- ability to meet effluent criteria requirements
- degree of system and operational complexity
- ability to handle varying flow conditions
- capital and operating costs
- reliability and operating experience in similar applications and climate
- solids and biosolids management.

Based on the results of the initial review summarized in Tables 20 and 21, the secondary treatment processes to be carried forward for further consideration during the preliminary design phase are:

- Conventional Activated Sludge
- Extended Aeration
- Sequencing Batch Reactor

7.0 TERTIARY TREATMENT

Depending on receiving water constraints and regulatory requirements, low effluent total phosphorus levels (i.e., less than 0.3 mg/L) may be required at some wastewater treatment facilities. These very low limits require a tertiary level of treatment of the wastewater stream.

Currently, nearly all wastewater treatment facilities in Ontario provide a minimum of secondary treatment. In some receiving waters, the discharge of secondary effluent could still degrade water quality and, as a result, an advanced level of treatment is required. There are several treatment technologies identified to meet stringent effluent limits, which can be extensions of conventional treatment concepts or physical-chemical separation techniques such as adsorption, flocculation/sedimentation and membranes. Filtration (media and cloth) and Ballasted Flocculation have been considered as part of this Class EA.

7.1 Deep Bed Continuous-Backwash Filters

Depth filtration involves the removal of particulate material suspended in a liquid by passing the liquid through a filter bed comprised of a granular media (Metcalf & Eddy, 2003). Although depth filtration is one of the principal unit operations used in the treatment of potable water, the filtration of effluents from wastewater treatment processes is becoming more common. There are several different types of depth filtration technologies, including conventional downflow filters, deep-bed downflow filters, deep-bed upflow continuous-backwash filters, pulse bed filters and travelling bridge filters. The deep-bed continuous-backwash filters will be considered here.

The Dynasand® system (deep-bed continuous-backwash) is a unique filtration system that is used as a polishing system for final effluent. It continuously filters wastewater passing through the system, while continuously cleaning the filter media. The influent suspension is introduced into the filter through a feed nozzle and the flow enters a series of vertical tubes that protrude into the filter. These riser tubes discharge beneath the inlet distribution hood, which allows even distribution of the influent into the sand bed. The wastewater is filtered upward through the sand and the clean filtrate exits the unit over the filtrate weir at the top of the tank.

For the continuous cleaning of the media, a small supply of air is continuously introduced into the bottom of the tank into an airlift type system that extends to the top of the unit. The filtered solids and sand are pulled into the airlift and conveyed to the top of the unit to a washer. The impurities that are carried upwards and leave the filter via a washwater outlet and the purified sand sink down to the sand bed.

The Dynasand® filter system is capable of achieving phosphorous levels that are below 0.2 mg/L. Table 22 presents estimated footprint requirements for a Dynasand® filter system, based on supplier information (H2Flow, 2006).

Table 22: Estimated Dynasand® Filter Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Depth Filtration	20 m^2 (1) (2 Modules, 2 Cells/Module - 4 Cells)
Height Requirements	6 m (1)

1 Dimensions based on supplier information (H2Flow, 2006)

Some current installations in Ontario include Wasaga Beach Wastewater Treatment Plant (15,433 m^3/day), Elmvale Wastewater Treatment Plant (1,512 m^3/day), Sutton Wastewater Treatment Plant (3,412 m^3/day) and Bradford West Gwillimbury (11,146 m^3/day).

7.2 Low Head Travelling Bridge Filter

The low head travelling bridge filter technology is a continuous downflow system equipped with an automatic backwash cycle. This type of granular media gravity filter concept has been used successfully for decades in the water treatment industry. The low head filters typically have single or dual filter mediums, usually consisting of sand, anthracite or a combination of both.

A low head travelling bridge filter is normally divided horizontally into long independent filter cells. Treated wastewater flows through the medium by gravity and exits through an underdrain network. Each cell is backwashed individually by an overhead, travelling-bridge assembly, while all other cells remain in operation. Water used for backwashing is pumped up through the medium and deposited in a backwash trough. During the backwash, wastewater is filtered continuously through the cells still in operation (Metcalf & Eddy).

The low head travelling bridge filter system is capable of achieving phosphorous levels that are below 0.2 mg/L. Table 23 presents estimated footprint requirements for a low head travelling bridge filter system based on supplier information (H2Flow, 2006).

Table 23: Estimated Low Head Travelling Bridge Filter Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Low Head Filtration	40 m^2 (1)
Height Requirement	Low height requirement

1 Dimensions based on supplier information (H2Flow, 2006)

Some current installations of a low head travelling bridge filters in Ontario include Kemptville Wastewater Treatment Plant and Bala Wastewater Treatment Plant.

7.3 Surface Filtration

Surface filtration involves the removal of particulate material suspended in a liquid by mechanical sieving by passing the liquid through a thin filter material (Metcalf & Eddy, 2003). The AquaDisk® filter system will be considered as part of this review.

The cloth media is completely submerged during filtration. Solids are deposited on the outside of the cloth as the wastewater flows by gravity through the module. As solids accumulate on and within the media, a mat is formed and the liquid level in the tanks or basin increases. The filtered liquid enters the internal portion of the disk where it is directed to final discharge through the centre shaft.

Predetermined timing or increased headloss initiates periodic backwashing. During backwashing, a pump provides suction to the vacuum heads, allowing solids to be vacuumed from the cloth. During backwash, disks rotate slowly, allowing each segment to be cleaned. Backwash water is directed back to the headworks and filtration is not interrupted during this cycle.

The AquaDisk® filter system is capable of achieving phosphorous levels that are below 0.2 mg/L. Table 24 presents estimated footprint requirements for an AquaDisk® filter system based on supplier information (Envirocan, 2006).

Table 24: Estimated AquaDisk® Cloth Filter Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Surface Filtration	25 m^2 (1) (2 Modules, 4 Disks/Module - 8 Disks)
Height Requirements	4 m (1)

1 Dimensions based on supplier information (Envirocan, 2006)

The only example of a current installation in Ontario is the Tilbury Water Pollution Control Plant (similar size to projected flows at Mississippi Mills).

7.4 Ballasted Flocculation

As with the depth filtration process, the flocculation/sedimentation process typically used in the water treatment industry can also be used for polishing of wastewater effluent from the secondary treatment process. One system used in Ontario in recent years is the Actiflo® ballasted flocculation process. Actiflo® can meet or exceed very high water quality standards with high removal rates for suspended solids, colour and phosphorous.

In this process, the secondary effluent is conveyed to a flash mix chamber where coagulant is introduced to destabilize the suspended solids (coagulation). The coagulated wastewater then moves through an injection chamber, where polymer and microsand are added. The floc attaches to a microsand coated with a polymer in a maturation chamber. Finally, the flocculated wastewater passes through a lamella type clarifier, where the ballasted floc settles and clarified effluent is conveyed to the disinfection process.

Settled sludge and microsand is continuously pumped from the bottom of the clarifier and recycles through hydrocyclone concentrators, where the microsand is separated from the sludge. The separated sludge is sent to waste and the microsand is recycled back to the process.

The Actiflo® system is capable of achieving phosphorous levels that are below 0.2 mg/L. Table 25 presents estimated footprint requirements for an Actiflo® system based on supplier information (John Meunier, 2006).

Table 25: Estimated Actiflo® Footprint Requirements

PROCESS	FOOTPRINT REQUIREMENTS
Actiflo®	20 m^2 (1) (2 Modules)
Height Requirements	4 m (1)

1 Dimensions based on supplier information (John Meunier, 2006)

Current wastewater tertiary treatment installations in Ontario include the Lindsay Wastewater Treatment Plant (21,500 m^3/day) and the Deseronto Wastewater Treatment Plant (1,600 m^3/day). There are also a number of water treatment plant installations, including Carleton Place and Renfrew Water Treatment Plants.

7.5 Summary of Costs for Tertiary Treatment Alternatives

Table 26 provides a summary of comparative capital costs of the tertiary treatment technologies. The estimated footprint and height requirements are generally the same and are reflected in the building costs.

Table 26: Cost Summary for Tertiary Treatment Alternatives

DESCRIPTION	Depth Filter	Low Head Filter	Surface Filter	Ballasted Flocculation
Tertiary Treatment Building	$500,000	$500,000	$500,000	$500,000
Concrete Tank	$200,000	$250,000	N/R (1)	N/R (1)
Tertiary Treatment Equipment	$250,000	$250,000	$400,000	$600,000
Process Mechanical	$200,000	$200,000	$250,000	$350,000
HVAC Provisions	$100,000	$100,000	$100,000	$100,000
Electrical and Instrumentation	$100,000	$100,000	$100,000	$100,000
Miscellaneous	$250,000	$250,000	$250,000	$250,000
SUBTOTAL	**$1,600,000**	**$1,650,000**	**$1,600,000**	**$1,900,000**
Contingency (20%)	$320,000	$330,000	$320,000	$380,000
Engineering (15%)	$288,000	$297,000	$288,000	$342,000
TOTAL	**$2,208,000**	**$2,277,000**	**$2,208,000**	**$2,622,000**

1 Concrete tanks are not required. Equipment costing includes steel containment.

7.6 Evaluation of Tertiary Treatment Alternatives

Four tertiary treatment technologies have been evaluated based on technical requirements, depth filtration, surface filtration and ballasted flocculation. Table 27 summarizes the advantages and limitations of each alternative.

Table 27: Advantages and Limitations of Tertiary Treatment Alternatives

TECHNOLOGY	ADVANTAGES	LIMITATIONS
Depth Filtration	• Consistent removal efficiency over a wide flow range • Proven technology for plants of similar scale • Small footprint • Continuous filtration • Fully automated system	• Operating costs associated with aeration system • Moderately complex operation • Proprietary system may make maintenance difficult and does not foster competitive costing • Sand media replacement requirements
Low Head Filtration	• Consistent removal efficiency over a wide flow range • Relatively simple operation compared to all other options • Proven technology for plants of similar scale • Low head requirement • Backwash does not require storage, control valving and excessive pumping • Regular and short-duration backwashes reduce loading rates • Continuous filtration • Fully automated system • Lowest relative capital cost	• Largest footprint • Some history of mechanical failures with backwash carriage system
Surface Filtration	• Consistent removal efficiency over a wide flow range • Low backwash rates (< 2% influent flow) • Low head requirements for gravity feed • Small footprint • Life expectancy of disk media approximately 8-10 years	• Relatively limited examples of plants of similar scale and may require pilot testing • Proprietary system may make maintenance difficult and does not foster competitive costing

Ballasted Flocculation	• Consistent removal efficiency over a wide flow range • Proven technology for plants of similar scale • Small footprint	• Mechanically complex system • High operating and capital costs associated to mixers, chemical (i.e., polymer) and microsand requirements • Microsand replacement required • Intermediate pumping likely required

7.7 Tertiary Treatment Summary

Based on the advantages and limitations presented in Table 24, Filtration and Ballasted Flocculation are feasible options for implementation as part of the Almonte Ward Communal Sewage System. All three technologies are capable of providing the required level of treatment and are compatible with the listed secondary treatment alternatives.

The capital costs associated with the Actiflo® system are estimated to be higher than the other Options. The operations and maintenance costs for the Actiflo® system are also likely higher than the filtration systems due to chemical costs and complexity. The cloth filtration system is a relatively new technology and, as a result, may present some operational challenges. Intermediate pumping may be required for the depth filtration and ballasted flocculation systems. The travelling-bridge type filter is estimated to be lowest in capital cost relative to the other filter technologies presented herein.

Based on a review of the available tertiary treatment technologies, all three filter type technologies meet the requirements of the project and can be considered during preliminary design.

8.0 DISINFECTION

Disinfection is considered to be the primary mechanism for the inactivation and destruction of pathogenic organisms to prevent the spread of waterborne diseases to downstream users and the environment. It is important that wastewater be adequately treated prior to disinfection in order for any disinfectant to be effective.

Two common methods of disinfection of wastewater are Chlorination/Dechlorination and Ultraviolet (UV) Disinfection. Both disinfection technologies can effectively meet the discharge requirements for treated wastewater. Other disinfection technologies include ozone and gamma ray irradiation, however, these technologies have high operating and capital costs, are difficult to operate, and are not typically used in wastewater treatment applications. For these reasons, they have not been considered as part of this Class EA.

8.1 Chlorination/Dechlorination

The most commonly used disinfectant in wastewater treatment has historically been chlorine. Chlorine is relatively inexpensive, efficient, dependable, and easy to handle, and because of its penetrating ability, will attack microorganisms readily. Chlorine can be applied to wastewater as a gas or liquid. However, any free (uncombined) chlorine remaining in the wastewater, even at low concentrations, is highly toxic to aquatic life in the receiving water body (EPA, 2004). Therefore, removal of even trace amounts of free chlorine by dechlorination is often needed to protect fish and other aquatic life.

Dosage and contact times are the two governing factors used to optimize the chlorine disinfection process. According to MOE Guidelines, a chlorine residual of 0.5 mg/L is generally required for effective disinfection at a contact time of 30 minutes at average day flows, and 15 minutes at peak flows (MOE, 1984). When calculating the required contact time, it is important to consider the dimensions of the contact chamber, since the geometry influences the contact time. Furthermore, the disinfection effectiveness of chlorine is greatly enhanced by effectively mixing of the wastewater and chlorine.

In order to produce non-toxic effluent prior to discharge, any remaining chlorine residual must be neutralized by the addition of a Dechlorination agent (e.g., sodium bisulfite). The dechlorination process occurs rapidly and, as a result, additional tankage is not required.

Chemicals required for the Chlorination/Dechlorination process require careful consideration for storage and handling to reduce related health and safety concerns.

The required contact tank volume required in the case of the Almonte Ward Communal Sewage System is approximately 180 m^3 for a contact time of 15 minutes at instantaneous peak flows of 18,000 m^3/day. The system will also require bulk storage, chemical pumping and control equipment for dosing.

8.2 Ultraviolet (UV) Disinfection

An Ultraviolet (UV) Disinfection system transfers electromagnetic energy from a mercury arc lamp to an organisms genetic material, ultimately destroying the cell's ability to reproduce (EPA, 1999). The effectiveness of a UV disinfection system depends on the characteristics of the wastewater, the intensity of UV radiation, the amount of time the microorganisms are exposed to the radiation, and the reactor configuration (EPA, 1999).
The main components of a UV disinfection system are lamps, reactors, ballasts and cleaning components. The source of UV radiation can be delivered using low-pressure (output wavelength of 253.7 nm) or medium pressure lamps (output wavelengths from 180 to 1,370 nm). For intermittent processes, consideration must be given for flow equalization before UV disinfection.

UV disinfection units are typically installed within a channel and equipped with an automatic cleaning system. UV disinfection is becoming more popular for new wastewater treatment facilities in Ontario due to their relatively safe and easy operation. One disadvantage is the relative amount of energy required to operate as compared to a chlorination/dechlorination facility.

8.3 Summary of Costs for Disinfection Alternatives

Table 28 provides a summary of comparative capital costs both disinfection alternatives.

Table 28: Cost Summary for Disinfection Alternatives

DESCRIPTION	Chlorination/Dechlorinatio n	UV Disinfection
Chemical Building	$150,000	$75,000
Contact Tank or Channels	$100,000	$100,000
Disinfection Equipment	$100,000	$100,000
Process Mechanical	$50,000	$50,000
HVAC Provisions	$75,000	$50,000
Electrical and Instrumentation	$50,000	$50,000
Miscellaneous	$75,000	$75,000
SUBTOTAL	**$600,000**	**$550,000**
Contingency (20%)	$120,000	$110,000
Engineering (15%)	$108,000	$99,000
TOTAL	**$828,000**	**$759,000**

Total costs of UV disinfection (i.e., including annual operating costs) can be competitive with chlorination when the dechlorination step is included. The annual operating costs of UV disinfection include power consumption; miscellaneous equipment repairs (approximately 2.5% of total equipment costs); replacement of lamp, ballasts and sleeves; and staffing requirements. Annual operating costs of a chlorination and dechlorination system include minimal power consumption; miscellaneous equipment repairs; chemical costs; and staffing requirements.

Generally, the application of chlorination and dechlorination facilities occurs where a contact chamber exists. It is understood that, while the initial costs associated with chlorination and dechlorination are higher than UV, the life-cycle costing must be considered.

8.4 Evaluation of Disinfection Alternatives

Table 29 compares the advantages and limitations of the grit removal technologies outlined above.

Table 29: Advantages and Limitations of Disinfection Alternatives

TECHNOLOGY	ADVANTAGES	LIMITATIONS
Chlorination/ Dechlorination	• Chlorination and dechlorination is an effective way of disinfecting wastewater effluent • Proven technology at similar scales and climate • Dechlorination protects aquatic life from toxic effects of chlorination	• Chemical dechlorination can be difficult to control when near zero level of residual chlorine are required • Significant overdosing of sulfite can lead to sulfate formation, supressed dissolved oxygen content and lower the pH of the final effluent • Presents operational risks associated with handling, transporting and storing potentially hazardous chemicals • Operationally more complex than UV disinfection • No existing contact tank
UV Disinfection	• UV is an effective way of disinfecting wastewater effluent • Proven technology at similar scales and climate • UV is a physical process rather than a chemical disinfectant, which eliminates the need to handle, transport and store potentially hazardous chemicals • UV is relatively easy to operate compared to chlorination and dechlorination • UV is generally an automated system with safeguards to ensure optimized operation • No residual effect that can be harmful to aquatic life • UV requires a shorter contact time than other disinfectants • Small footprint requirements	• Requires chemical cleaning • Lamp replacement causes maintenance costs to increase • Low dosages may not effectively inactivate viruses and cysts • Lamp fouling issues • Turbidity and suspended solids in the wastewater can render the disinfection process ineffective • UV is not as cost-effective as chlorination and dechlorination over a long period of time

8.5 Disinfection Summary

Based on the review of both Disinfection Options, it is recommended that UV disinfection be carried forward for further review. Although Chlorination/Dechlorination presents itself as a proven and cost-effective way of disinfecting wastewater effluent, it is understood that this technology is complex and presents a hazardous risk for humans (i.e., handling, transport and storage) and aquatic life.

9.0 SLUDGE AND BIOSOLIDS HANDLING

The solids generated by the wastewater treatment process (i.e., the liquid train) are typically referred to as sludge. The handling and treatment of sludge is a significant consideration in the design and operation of a sewage treatment plant Two basic goals of treating the sludge are to reduce the volume and stabilize the organic material. The stabilized material is often referred to as biosolids. The ultimate plan for disposal of the biosolids is a very important factor in the design of a sewage treatment plant.

There are a number of Unit Operations that can make up the sludge and biosolids handling process at a sewage treatment plant. These can include the following:

- Thickening
- Digestion
- Dewatering
- Storage and Disposal

In the case of the Almonte STP, there are two possible scenarios that would generate different quantities of solids depending on the selection of a secondary treatment process. As previously noted, processes that include primary clarifiers, such as a Conventional Activated Sludge (CAS) process, tend to generate more sludge than an Extended Aeration (EA) process that does not utilize primary clarifiers. Estimated quantities of solids produced from the wastewater treatment process for both possibilities are as follows:

- Conventional Activated Sludge: 1000 kg/day
- Extended Aeration/Sequencing Batch Reactor: 800 kg/day

These quantities are based on the estimated strength and loading of the raw sewage, along with assumptions regarding the efficiency of the wastewater treatment process. It should be noted that quantities of solids can also be reduced if digestion processes are used. Digestion converts solids to gases and liquids. Therefore, the total quantity of solids for ultimate disposal can be reduced.

9.1 Thickening

Thickening of the sludges generated by the primary and secondary treatment processes is often undertaken to reduce the volume of waste generated. This is done by removing liquid from the sludge, and thereby increasing the thickness or percent concentration of solids of the sludge. Reducing the volume of the sludge can significantly decrease the required size, and thereby cost of downstream infrastructure (e.g. digesters, pumps, piping, etc.). It can also lower the operational costs associated with handling the sludge (e.g. storage, hauling, energy).

9.1.1 Technologies

Thickening can be accomplished using various technologies, including, gravity settling (either co-thickened within primary clarifiers or in a separate settling tank); rotary drum thickening; gravity belt thickening, dissolved air flotation, and centrifuging. Sludges can typically be thickened from 0.5 to 1.5% to over 5% concentrations, depending on the technology utilized. Gravity thickening systems tend to provide thinner sludges than mechanical systems but involve less complex operations and are lower in capital cost. Mechanical thickeners are typically used on biological sludges (i.e., those generated from the secondary treatment process) as opposed to primary sludges that are thickened in the primary clarifiers that they are generated in.

Centrifuges tend to be very high in capital cost relative to other technologies, are more suitable for larger facilities and for dewatering, and, therefore, have not been considered further for thickening. Dissolved Air Flotation (DAF) is high in operational cost, more suitable for larger facilities and can be operationally difficult, and, therefore, has not been considered further. The following technologies have been considered in the case of the Almonte STP:

- Gravity Belt Thickener: With gravity belt thickeners (GBTs), polymer conditioned sludge is distributed evenly across the width of a moving fabric belt. Free water drains through the belt, while suspended solids are retained on the surface. Rows of plough blades ride on the belt surface and turn the sludge to release additional water and expose clear areas on the belt for improved drainage. A high pressure wash is used to clean polymer and suspended solids from the pores of the fabric belt. Filtrate is collected and returned to the head of the plant. Sludge can be thickened to over 5% solids.

- Rotary Drum Thickening: With rotary drum thickeners (RDTs), polymer conditioned sludge is fed into one end and is distributed onto the internal surface of a rotating drum screen. Flocculated sludge solids are retained on the inner surface, while free water drains through the screen. Filtrate is collected in a trough and is returned to the head of the plant. Sludge solids are conveyed towards the outlet end of the drum by flights or an internal screw conveyor. The inside and outside drum surfaces are periodically rinsed to flush trapped solids from the screen. Sludge can be thickened to over 5% solids.

- Gravity Thickening with Polymer Addition: Gravity thickening (GT) in a tank is achieved in a similar fashion as sedimentation in a conventional clarifier. Dilute sludge is dosed with polymer and fed to a thickening tank where it is allowed to settle and compact. Supernatant is drawn off the top of the tank and thickened sludge is pumped from the bottom of the unit. Gravity thickening is capable of achieved concentration of about 2-4% for both raw and digested Waste Activated Sludge (WAS).

9.1.2 Technology Comparison

Both GBTs and RDTs have similar advantages and disadvantages. Each method is capable of producing a higher concentration of biosolids than Gravity Thickening, but has higher capital and operating costs. GBT and RDT are more suitable for thickening digested biosolids, where a sludge concentration of up to 5% is acceptable. At higher total solids concentrations, pumping and aeration operations become inefficient. By thickening following digestion, the size of digestion tanks will not be reduced, but the size of storage tanks will be reduced.

For GT, the maximum total solids concentrations achievable is 4%, making it suitable for pre-thickening prior to digestion, as thermophilic conditions will not be created. By pre-thickening prior to digestion, digesters can be reduced.

9.1.3 Possible Options

Except for the case of co-thickening in primary clarifiers, another unit operation does entail additional capital and operational costs. Co-thickening can only occur if primary clarifiers are available, which depends on the selection of the secondary treatment process. For example, if a Conventional Activated Sludge process is selected for secondary treatment, then co-thickening can be considered. If an Extended Aeration or

SBR process is selected for secondary treatment, then an additional thickening process would be required if thickening were desirable.

In the case of the Almonte STP, it has been assumed that, for a CAS process, either co-thickening in the primary clarifiers could be undertaken, or a separate thickening process could be utilized for the WAS (i.e., the sludge generated from the secondary treatment process). This would be undertaken to reduce the necessary volume of the digesters. For EA and SBR processes, it has been assumed that thickening would be undertaken using a separate gravity settling tank with polymer addition. This would be used to thicken sludge in order to reduce the required sludge of the digester. The following options have been identified in the case of the Almonte STP:

Option 1: CAS	Option 1A: No thickening
	Option 1B: Co-thickening of Secondary in Primary Clarifiers
	Option 1C: Separate thickening of Secondary Sludge
Option 2: EA	Option 2A: No thickening
	Option 2B: Separate thickening of Secondary Sludge
Option 3: SBR	Option 3A: No thickening
	Option 3B: Separate thickening of Secondary Sludge

Table 30 presents an estimate of capital costs associated with each option (gravity thickening has been assumed as the preferred technology).

Table 30: Thickening Options - Summary of Capital Costs

Item	Option 1A[1]	Option 1B	Option 1C	Option 2A[1]	Option 2B	Option 3A[1]	Option 3B
Tankage/Building	$600,000	$0	$150,000	$400,000	$150,000	$400,000	$150,000
Mechanical Equipment	$200,000	$0	$250,000	$100,000	$200,000	$100,000	$200,000
Process Piping	$100,000	$150,000	$100,000	$50,000	$100,000	$50,000	$100,000
Electrical	$100,000	$50,000	$75,000	$50,000	$75,000	$50,000	$75,000
Instrumentation	$50,000	$25,000	$50,000	$50,000	$50,000	$50,000	$50,000
Civil Works	$150,000	$25,000	$50,000	$100,000	$50,000	$100,000	$50,000
Miscellaneous	$100,000	$50,000	$25,000	$50,000	$25,000	$50,000	$25,000
SUBTOTAL	**$1,300,000**	**$300,000**	**$700,000**	**$800,000**	**$650,000**	**$800,000**	**$650,000**
Contingency (20%)	$260,000	$60,000	$140,000	$160,000	$130,000	$160,000	$130,000
Engineering (15%)	$234,000	$54,000	$126,000	$144,000	$117,000	$144,000	$117,000
TOTAL	**$1,794,000**	**$414,000**	**$966,000**	**$1,104,000**	**$897,000**	**$1,104,000**	**$897,000**

[1] Estimated cost increase for additional digester and storage tankage and equipment

As illustrated, in all cases, the option of No thickening results in significant additional capital costs associated with digesters. If a CAS process is ultimately chosen during preliminary design, then co-thickening is recommended as the most economical option. If an EA or SBR process is selected, then thickening prior to digestion is recommended. Although all thickening technologies can be considered further during preliminary design, GT is likely the least costly in terms of both capital and operating costs.

9.2 Digestion

Sludge digestion is a biological process in which organic solids (there is a certain quantity of inert solids that cannot be reduced) are decomposed into stable substances (referred to as biosolids). Digestion reduces the total mass of the solids (by converting solids to liquids and gases), destroys pathogens, and makes it easier to dewater the biosolids. Stabilized sludge, or biosolids, are largely inoffensive. Sludge digestion can be accomplished either anaerobically (in the absence of oxygen) or aerobically (in the presence of oxygen).

9.2.1 Technologies

- Anaerobic Digestion: In anaerobic digestion, a two-stage system is usually utilized in which organics are metabolized by bacteria. In the first stage, the sludge is heated and mixed in a closed tank for approximately 15 days. The sludge is then conveyed to a second tank that serves primarily as storage and settling and decanting of liquid. As the organics are broken down by anaerobic bacteria, carbon dioxide and methane gas are formed. The methane is used as a fuel to heat the first digestion tank, as well as generate electricity in boilers for other plant uses. Anaerobic digesters require careful operation and are typically costly to construct.

- Aerobic Digestion: In aerobic digestion, sludge is aerated in an open tank for about 20 days. Methane gas is not formed in this process. While less expensive to construct, aerobic digestion is more costly to operate, due to the blower energy required. It also cannot be used to generate electricity to power the plant.

9.2.2 Comparison of Technologies

Anaerobic digestion entails more capital cost than aerobic digestion, however, operating costs can be lower and energy produced from the process can be used to produce electricity to offset plant energy usage. Anaerobic digesters are more complex to operate than aerobic digesters and typically involve more maintenance and operator attention. Because anaerobic digesters are associated with CAS processes, which include primary sludges, the treated biosolids are typically easier to dewater than those associated with aerobic digesters. While aerobic digesters are significantly less costly to construct, they do require additional energy to operate. They are more appropriately suited for use with EA processes.

9.2.3 Possible Options

In the case of the Almonte STP, the following Options have been identified:

- **Option 1: CAS - Anaerobic Digestion**
- **Option 2: SBR/EA - Aerobic Digestion**

In both cases, thickening of the sludge prior to digestion has been assumed. As previously discussed, this will serve to reduce the size of the tankage and equipment associated with digestion. Gravity Thickening has been assumed to increase sludge to about 3% solids prior to digestion.

In the case of Option 1, a total primary digester volume of 800 m^3 is estimated to be required, based on a 20-day retention time. A secondary digester of equal size has been assumed for redundancy purposes. The secondary digester would typically provide opportunity for further sludge thickening and some storage of the biosolids. Based on this size, two tanks nominally sized at 12.5 m with 6.5 metre side water depth have been assumed.

In the case of Option 2, a total digester volume of 1,200 m^3 is estimated to be required based on a 40-day solids retention time (SRT). Based on this size, a single tank, nominally sized at 10 m x 20 m x 6 metre side water depth has been assumed. One advantage of an aerobic digester is the possible future conversion to an aeration tank for possible plant capacity expansion. Therefore, sizing it similar to the aeration tanks provides some benefit. Table 31 provides a comparison of capital costs associated with digestion.

Table 31: Digestion Options - Summary of Capital Costs

Item	Option 1 Anaerobic Digestion	Option 2 Aerobic Digestion
Tankage/Building	$900,000	$400,000
Mechanical Equipment	$300,000	$100,000
Process Piping	$300,000	$50,000
Electrical	$150,000	$50,000
Instrumentation	$100,000	$50,000
Civil Works	$200,000	$100,000
Miscellaneous	$50,000	$50,000
SUBTOTAL	**$2,000,000**	**$800,000**
Contingency (20%)	$400,000	$160,000
Engineering (15%)	$360,000	$144,000
TOTAL	**$2,760,000**	**$1,104,000**

9.3 Dewatering

Dewatering of the biosolids usually occurs after digestion, although undigested sludge can also be dewatered if digestion is not a part of the process. Dewatering is used to further reduce the water content of the biosolids to a concentration of between 20 to 40%, depending on the type of biosolids and dewatering method used. Typical dewatering methods include centrifuging, belt filter presses, rotary presses, and sludge drying beds.

9.3.1 Technologies

- Centrifuge: Centrifuges utilize a high speed process that uses the force of rapid rotation of a cylindrical (conical) bowl to separate wastewater solids from liquid.

- Belt Filter Press: A belt filter dewaters by applying pressure to the biosolids to squeeze the water out. Biosolids sandwiched between two tensioned power belts are passed over and under rollers of various diameters. Increased pressure is created as the belts are passed over and under rollers of varying diameters.

- Rotary Press: A rotary press slowly rotates the sludge through a circular conduit while pressure is applied to the discharge point. Filtrate or supernatant is squeezed out and a cake is discharged from the press.

- Sludge Drying Beds: Sludge drying beds are used to store sludge over large areas. Supernatant is typically collected in an under drain system and recycled back to the plant. Dewatered sludge can be removed after several weeks of drying.

9.3.2 Comparison of Technologies

Centrifuges are relatively common for dewatering applications due to their high performance and relatively low maintenance. They can result in lower odours and are easy to clean and maintain. Disadvantages include high power consumption, special structural considerations (due to the speed that they operate at) and long start-up times. Conversely, Belt filter presses tend to result in more odours, difficult cleaning and lower performance, although start-up is quicker and they are quieter than centrifuges.

A rotary press offers the advantage of a small footprint, as well as low power costs and slow moving equipment. The main disadvantage with Rotary Presses is that they are very suitable for primary sludge, however, their performance is low compared to other technologies.

Sludge Drying Beds are very economical, requiring very little operator attention. They do, however, require a large footprint and odour issues can occur.

9.3.3 Possible Options

In the case of the Almonte STP, the following options have been identified:

- **Option 1: Mechanical Dewatering**
- **Option 2: Drying/Disposal in Lagoon**

Table 32 presents an estimate of capital costs associated with each option.

Table 32: Dewatering Options - Summary of Capital Costs

Item	Option 1 Mechanical Dewatering	Option 2 Drying/Disposal in Lagoon
Tankage/Building	$150,000	$25,000
Equipment	$400,000	$100,000
Process Piping	$50,000	$25,000
Electrical	$50,000	$25,000
Instrumentation	$20,000	$5,000
Civil Works	$80,000	$180,000
Miscellaneous	$50,000	$0
SUBTOTAL	**$800,000**	**$360,000**
Contingency (20%)	$160,000	$72,000
Engineering (15%)	$144,000	$64,800
TOTAL	**$1,104,000**	**$496,800**

9.4 Storage and Disposal

The plan for storage and ultimate disposal of sludge or biosolids is an important consideration when determining plant infrastructure requirements. Historically, biosolids have largely been land applied as nutrients. Other disposal methods can include landfilling and incineration.

For land application in Ontario, biosolids must be stabilized in accordance with the requirements of the Nutrient Management Act (NMA), Ontario Regulation 267/03 (O.Reg. 267/03) enacted under the NMA and the MOE Design Guidelines. Under O.Reg. 267/03, the *E.Coli* counts in the stabilized biosolids must not exceed 2 million per gram of dry solids. There is currently no benefit in Ontario to producing a better quality biosolids with lower pathogen counts. All the criteria for land application rates and storage requirements apply, provided that the *E.Coli* criterion is met.

The on-site storage of treated biosolids is necessary if final disposal methods are not available during certain times of the year. For example, if the only method of disposal is to land apply the biosolids, O. Reg 267/03 stipulates that 240 days of storage must be provided. This is because land application is not permitted year round. If other methods are available (e.g., landfilling), then on-site storage can be reduced or eliminated. Sludge can be stored in a liquid or dewatered form. If landfilling is all or a part of the intended disposal method, then dewatering is necessary such that the material can be considered a solid waste.

9.4.1 Technologies

Typical technologies for on-site storage can include tankage (e.g., glass fused to steel, concrete) and lagoon basins. If the sludge is dewatered, then some form of storage pad with minimal covered enclosure can be considered.

Disposal technologies can include land application, landfilling, incineration and storage in lagoons. Incineration has not been considered as there are no known applications in Ontario.

9.4.2 Technology Comparison

Land application has, historically, been the preferred technology for many municipalities. As Almonte is largely in a rural setting, this option provides some benefits. However, with the enactment of the Nutrient Management Act (NMA), it may be more difficult to secure licensed fields in the future. If land application is part or all of the disposal option, then stabilization is required.

In the case of Almonte, the potential of utilizing the existing lagoons presents a possible economically attractive alternative for disposal. In order to rehabilitate the lagoon, the sludge could be utilized (i.e., rather than hauling to farmers' fields, the biosolids could be utilized over time to reclaim the lagoons). The sludge could either be stored in the lagoons without stabilization, or it could be stabilized, possibly to reduce odours. This would eliminate the need to haul off-site and, at the same time, provide a long term plan for rehabilitating the lagoons.

Landfilling dewatered sludge is a possible option. However, this would definitely require dewatering, adding to capital cost, as well as hauling to a licensed landfill.

9.4.3 Possible Options

Storage and disposal options available for the Almonte STP include the following:

- **Option 1: Store on site and land apply**
- **Option 2: Landfill year-round**
- **Option 3: Store on site and land apply and landfill during portions of the year**
- **Option 4: Dispose in existing Lagoon**

Option 1: This option does not necessarily require sludge dewatering but would definitely require stabilization, as well as significant on-site storage. In the case of a CAS process and anaerobic digestion, the storage requirements for 240 days are estimated to be in the order of 3,100 m^3. If dewatering is not practiced and it is assumed that 50% of the secondary digester capacity can be used for storage, then two storage tanks nominally sized at 15 m diameter with 8 m sidewater depth are assumed. In the case of an EA process, slightly smaller tanks are assumed (12.5 m diameter with 8 m sidewater depth).

Option 2: This option definitely requires on-site dewatering but would not require significant storage. Some nominal on-site storage of the dewatered cake could be provided (say one week). In addition, digestion would not necessarily be required, since landfilling would be the final disposal option (i.e., expending the energy and resources required for digestion when ultimate disposal is in a landfill may not be economical). For this option, limited on-site storage would be required.

Option 3: This option could reduce the required on-site storage requirements if biosolids are land applied during appropriate times of the year (periods during Spring to Fall) and landfilled during winter periods. This option assumes that storage requirements are reduced by about two-thirds (say 160 days vs 240 days). On-site tankage costs would be significantly lower.

Option 4: This option would involve utilizing the existing lagoons to dispose of the sludge or biosolids (i.e., if stabilized). In this case, digestion and dewatering may not be required, since final disposal would be within the existing Lagoon. The Lindsay STP utilizes this strategy. However, another option may be to proceed with digestion and possibly dewatering and use the material for rehabilitation of the Lagoon area over several years. This would at least eliminate hauling the material off-site and it would serve to provide a viable material that could be used to slowly infill the Lagoon. Based on existing available volumes in the Lagoon (i.e., accounting for sludge accumulation) it is estimated that there is over 50 years of storage available within the Lagoon if the sludge is not dewatered. Dewatering may not be practical in this case.

Table 33 presents an estimate of capital costs associated with each option.

Table 33: Sludge Storage/Disposal Options - Summary of Capital Costs

Item	Option 1 Store on site and Land Apply	Option 2 Landfill Year-Round	Option 3 Land apply and Landfill	Option 4 Dispose in Lagoon
Tankage/Building	$800,000	$50,000	$500,000	$0
Mechanical Equipment	$200,000	$50,000	$100,000	$50,000
Process Piping	$150,000	$25,000	$100,000	$50,000
Electrical	$75,000	$25,000	$50,000	$25,000
Instrumentation	$25,000	$0	$25,000	$0
Civil Works	$250,000	$25,000	$150,000	$150,000
Miscellaneous	$100,000	$25,000	$75,000	$25,000
SUBTOTAL	**$1,600,000**	**$200,000**	**$1,000,000**	**$300,000**
Contingency (20%)	$320,000	$40,000	$200,000	$60,000
Engineering (15%)	$288,000	$36,000	$180,000	$54,000
TOTAL	**$2,208,000**	**$276,000**	**$1,380,000**	**$414,000**

9.5 Summary of Biosolids Handling

The managing of the “solids stream” from a sewage treatment plant is just as important as the liquid stream. The need for various solids handling operations is interdependent on the liquid stream processes.

There are a number of technologies capable of accomplishing the same goals and their selection depends on site specific considerations, as well as capital and operating costs. In many cases, there is no measurable difference in environmental impact between technologies.

It is recommended that thickening of secondary sludge be included in the preferred design concept. How thickening is achieved will depend on the selection of the secondary treatment process, as well as more detailed cost comparisons during the preliminary design stage. Preliminary review suggests that Gravity Thickening offers some capital and operating cost advantages over Mechanical thickening.

The selection of a stabilization method depends on the secondary treatment process selected. The option of not digesting at all is available if storage of sludge within the Lagoon is considered acceptable.

The need for dewatering depends on the ultimate disposal method. For example, there are some benefits associated with using the existing Lagoon for disposal. In this case, dewatering would not be required. However, if landfilling is chosen, then dewatering will definitely be required. Based on the overall evaluation, it is likely that dewatering will not be required. However, further review during preliminary design is recommended. Provisions for installation of dewatering equipment in the future could be incorporated (i.e., allowing for sufficient room for new facilities).

It is understood that the option of utilizing the existing Lagoon for ultimate disposal of the sludge or biosolids is economical. The securing of licensed fields for spreading or the need to haul to a landfill would be avoided. Stabilization could be undertaken even if the Lagoon is utilized in order to provide a better quality product for rehabilitation. Another option may be to use lime stabilization if the Lagoon is used for disposal. This could avoid the use of excessive energy but, at the same time, provide some stabilization.

10.0 SUMMARY AND EVALUATION OF DESIGN CONCEPTS

Previous Sections of this Report have examined various candidates for treatment technologies for liquid treatment and sludge management programs. Based on the evaluation of the design concepts for each of the unit operations, three preferred Design Concepts have been identified for the Almonte Ward Communal Sewage System and are presented in Table 34. Key considerations identified for the review and general comparison include:

- ability to meet effluent criteria requirements
- degree of system and operational complexity
- ability to handle varying flow conditions
- capital and operating costs
- reliability and operating experience in similar applications and climate
- solids and biosolids management.

Table 34: Preferred Design Concepts for the Almonte Ward Communal Sewage System

DESIGN CONCEPT NO.1	DESIGN CONCEPT NO.2	DESIGN CONCEPT NO.3
• Collection • Sewage Pumping • Optional Septage Receiving • Screening • Grit Removal • Optional Equalization Storage • **Conventional Activated Sludge** • **Filtration** • **Ultraviolet Disinfection** • Sludge Conditioning • **Anaerobic Digestion** • Digested Sludge Dewatering • Sludge Storage	• Collection • Sewage Pumping • Optional Septage Receiving • Screening • Grit Removal • Optional Equalization Storage • **Extended Aeration** • **Filtration** • **Ultraviolet Disinfection** • Sludge Conditioning • **Aerobic Digestion** • Digested Sludge Dewatering • Sludge Storage	• Collection • Sewage Pumping • Optional Septage Receiving • Screening • Grit Removal • Optional Equalization Storage • **Sequencing Batch Reactors** • Equalization Storage • **Filtration** • **Ultraviolet Disinfection** • Sludge Conditioning • **Aerobic Digestion** • Digested Sludge Dewatering • Sludge Storage

There are no significant choices for Preliminary Treatment Options and, as such, the items in bold will be used to identify the Preferred Design Alternatives.

Process flow diagrams and conceptual site plans are illustrated by the following Figures:

- Design Concept No.1: Figure 6 Process Flow Diagram
  Figure 7 Conceptual Site Layout
- Design Concept No.2: Figure 8 Process Flow Diagram
  Figure 9 Conceptual Site Layout
- Design Concept No.3: Figure 10 Process Flow Diagram
  Figure 11 Conceptual Site Layout

The following Tables summarize the costs associated with each of the Preferred Design Concepts.

Table 35: Collection System Cost Summary

DESCRIPTION	DESIGN CONCEPT 1 Conventional Activated Sludge + Anaerobic Digestion	DESIGN CONCEPT 2 Extended Aeration + Aerobic Digestion	DESIGN CONCEPT 3 Sequencing Batch Reactor + Aerobic Digestion
Collection System	$ Defined by Town Annually	$ Defined by Town Annually	$ Defined by Town Annually

Table 36: Sewage Pumping Cost Summary

DESCRIPTION	DESIGN CONCEPT 1 Conventional Activated Sludge + Anaerobic Digestion	DESIGN CONCEPT 2 Extended Aeration + Aerobic Digestion	DESIGN CONCEPT 3 Sequencing Batch Reactor + Aerobic Digestion
Pumping Station Upgrades	$2,000,000	$2,000,000	$2,000,000
Forcemain	$600,000	$600,000	$600,000
SUBTOTAL	**$2,600,000**	**$2,600,000**	**$2,600,000**
Contingency (20%)	$520,000	$520,000	$520,000
Engineering (15%)	$468,000	$468,000	$468,000
TOTAL	**$3,588,000**	**$3,588,000**	**$3,588,000**

Table 37: Sewage Treatment Cost Summary

DESCRIPTION	DESIGN CONCEPT 1 Conventional Activated Sludge + Anaerobic Digestion	DESIGN CONCEPT 2 Extended Aeration + Aerobic Digestion	DESIGN CONCEPT 3 Sequencing Batch Reactor + Aerobic Digestion
Optional Septage Receiving	$1,500,000	$1,500,000	$1,500,000
Screening	$400,000	$400,000	$400,000
Grit Removal	$550,000 (1)	$550,000 (1)	$550,000 (1)
Optional Equalization Storage	$200,000	$200,000	$200,000
Secondary Treatment	$4,950,000	$4,000,000	$4,500,000
Tertiary Treatment	$1,600,000 (2)	$1,600,000 (2)	$1,600,000 (2)
UV Disinfection	$550,000	$550,000	$550,000
Sludge Thickening	$300,000	$650,000	$650,000
Sludge Digestion	$2,000,000	$800,000	$800,000
Sludge Dewatering	$360,000	$360,000	$360,000
Storage and Disposal	$300,000	$300,000	$300,000
Administration Building	$750,000	$750,000	$750,000
Decommissioning of Lagoon	$500,000	$500,000	$500,000
Site Works and Miscellaneous	$750,000	$750,000	$750,000
SUBTOTAL	**$14,710,000**	**$12,910,000**	**$13,410,000**
Contingency (20%)	$2,492,000	$2,582,000	$2,682,000
Engineering (15%)	$2,647,800	$2,323,800	$2,413,800
TOTAL	**$20,299,800**	**$17,815,800**	**$18,505,800**

1 Costing based on Vortex Mixing System

2 Costing based on Low Head Travelling Bridge System

In addition to the capital costs required to implement the above-referenced Design Concepts, annual Operating and Maintenance costs must be considered. It is estimated that, for all three systems, the Operating and Maintenance costs will be approximately **$475,000.00**.

11.0 RECOMMENDATIONS

It is recommended that:

- A Public Meeting presenting the results of work completed to date be held on August 31, 2006 in order to obtain additional input.
- The following Preferred Design Concepts be presented at the Public Meeting:

Design Concept No.1:	Conventional Activated Sludge Mechanical Treatment Plant with Filtration, UV Disinfection and Anaerobic Digestion
Design Concept No.2:	Extended Aeration Mechanical Treatment Plant with Filtration, UV Disinfection and Aerobic Digestion
Design Concept No.3:	Sequencing Batch Reactor Mechanical Treatment Plant with Filtration, UV Disinfection and Aerobic Digestion

- The Collection System and Sewage Pumping System Upgrades be implemented as required.
- The Draft Phase 3 Report be circulated to all Review Agencies and Project Stakeholders.
- The Class EA proceed to Phase 4 - Prepare Environmental Study Report (ESR), once Phase 3 is complete. This Phase will involve documenting Phases 1 through 3 of the Class EA within an ESR and filing the document for mandatory review by Agencies, Public and Stakeholders.

12.0 REFERENCES

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APPENDIX ‘A’

MEETING MINUTES

APPENDIX 'B'

PUBLIC CONSULTATION DOCUMENTATION

APPENDIX ‘C’

RECEIVING WATER ASSESSMENT CORRESPONDENCE

APPENDIX ‘D’

COLLECTION SYSTEM INVENTORY