Optimization Guidance Manual for Sewage Works

2010

OPTIMIZATION GUIDANCE MANUAL

FOR

SEWAGE WORKS

2010

WATER ENVIRONMENT ASSOCIATION OF ONTARIO

ONTARIO MINISTRY OF THE ENVIRONMENT

ENVIRONMENT CANADA

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TABLE OF CONTENTS

CHAPTER 7. OPTIMIZATION TO MITIGATE EXTRANEOUS FLOW IMPACTS

CHAPTER 8. SEWAGE PUMPING STATION OPTIMIZATION

CHAPTER 9. FLOW METERING AND SAMPLING

CHAPTER 10. PRELIMINARY TREATMENT

CHAPTER 11. PRIMARY CLARIFICATION

CHAPTER 12. BIOLOGICAL TREATMENT

CHAPTER 13. AERATION SYSTEMS FOR BIOLOGICAL TREATMENT

CHAPTER 14. SECONDARY CLARIFICATION

CHAPTER 15. TERTIARY TREATMENT PROCESSES

CHAPTER 16. PHOSPHORUS REMOVAL

CHAPTER 1

WHAT IS OPTIMIZATION AND WHY OPTIMIZE

CHAPTER 1

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1.1 WHAT IS OPTIMIZATION

In the 1980's and 1990's, designers, owners and operators of sewage works recognized that there were opportunities to optimize sewage works in order to reduce capital cost of expansions, improve the effluent quality produced by the works, and reduce the cost of energy, chemicals, sludge disposal and other operational requirements. Over the past 20 years, the concept of sewage works optimization has evolved from a single study undertaken prior to an expansion of the works to a process of continuous improvement or an operational philosophy that is championed by the operating authority at all levels. The same approach can be used for optimization of drinking water treatment systems or other infrastructure, although different techniques may apply.

Optimization of sewage works is an iterative process that includes the following four major steps as illustrated in Figure 1-1:

- Step 1: Clearly define the objectives of the optimization program;

- Step 2: Evaluate specific components of the sewage works to establish the baseline conditions and the processes or factors that limit the capacity or the performance of the existing works;

- Step 3: Develop and implement a study program aimed at mitigating the capacity or performance limiting factors; and

- Step 4: Conduct follow-up monitoring after upgrades or process changes have been implemented to assess and document the results.

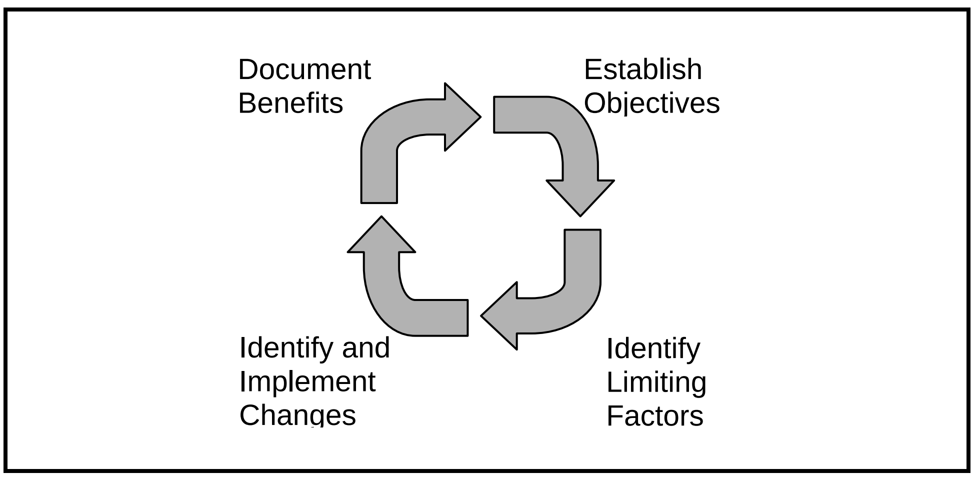

Figure 1-1 - Interactive Approach to Optimization of Sewage Works

(Adapted from FCM and NRC, 2003)

The specific details of the study program will depend on the optimization objectives. These objectives can be broadly-based, covering all aspects of the design and operation of the works, or can be narrowly focused on mitigating a specific problem. Optimization objectives might include the following, among others:

- Improving effluent quality to reduce the impacts of the sewage treatment plant discharge on the environment;
- Increasing the capacity of the works to service growth in the community;
- Upgrading the performance of the sewage treatment plant to meet more stringent regulatory requirements;
- Improving the reliability, flexibility and robustness of the works;
- Reducing the frequency of sewage bypass events and wet weather flow impacts on the works;
- Reducing the operating cost associated with energy, chemicals and labour;
- Reducing biosolids production and sludge management cost;
- Increasing anaerobic digester gas production for energy recovery; and/or
- Mitigating odour emissions from the works.

Often optimization of a sewage works to achieve one goal can result in improvements in other areas. For example, optimization to achieve lower chemical use and cost for phosphorus removal will also result in lower sludge production and lower sludge management costs. Similarly, improving the reliability and flexibility of the works can also result in improvements in effluent quality and reduced odour emissions.

Depending on the objectives of the optimization program, different approaches may be applicable. Table 1-1 (Nutt and Ross, 1995) presents some of the investigations that might be undertaken as part of an optimization project to address specific optimization objectives.

Table 1-1 - Activity and Objectives Matrix

ACTIVITY	OBJECTIVE			
	Performance Improvement	**Operating Cost Savings**	**Increased Capacity**	**Capital Cost Savings**
Hydraulic Analysis	✓		✓	✓
Individual Process Capacity Evaluation	✓	✓	✓	✓
Process Design Modifications	✓	✓	✓	✓
Process Control Modifications	✓	✓	✓	✓
Energy Audit		✓		
Operator Training Activities	✓	✓	✓	
I/I Control Study	✓	✓	✓	✓

Optimization methods will vary from sewage works to sewage works depending on program objectives and works design; however, some steps are common.

After the optimization objectives are established, the next step is to establish the baseline condition of the works or those components of the works that are of interest based on the objectives. This usually involves a desk-top analysis of historic data for a period of time that is representative of the current works design and operation, usually a minimum of three to five years.

A site visit is conducted in the accompaniment of operations and management staff. The key objectives of the site inspection are:

- To familiarize the optimization team with the design of the sewage works, the plant layout, and to identify the locations of significant sampling and monitoring stations;
- To obtain input from plant operations staff regarding equipment, hydraulic or process limitation in the plant based on their operating experience; and
- To discuss standard operating procedures for major unit processes.

The design of the works is compared to standard design practices and guidelines from references such as MOE Design Guidelines for Sewage Works (MOE, 2008), Ten State Standards (Great Lakes-Upper Mississippi River Board of State Public Health and Environmental Managers, 2004), Wastewater Engineering: Treatment and Reuse (Metcalf & Eddy, 2003), and Design of Municipal Wastewater Treatment Plants (WEF/ASCE, 1998).

A process capacity chart should be developed that identifies the capacity and capability of each unit process or the unit processes under investigation. This establishes the unit process or processes that limit the capacity or performance of the works. It will also serve to identify unit processes that would benefit from optimization and the field investigations that may be warranted.

Field investigations can then be undertaken to confirm the findings of the desk-top analysis and to identify the preferred method of optimizing the component of the works that is of interest. The specific field investigations undertaken will vary depending on the size of the works, the design of the works and the specific objectives of the optimization program.

The design or operational improvements are implemented and follow-up monitoring is undertaken to confirm the benefits.

A more detailed discussion of the historic data analysis and desk-top investigation is provided in Section 5.1 of this Guidance Manual. Specific field investigations that might be undertaken to confirm the findings of the desk-top study or to identify preferred optimization approaches are described in subsequent chapters of the Guidance Manual.

1.2 WHEN SHOULD AN OWNER/OPERATOR OPTIMIZE

In the United States, optimization of sewage treatment plants (STPs) became a priority when the United States (U.S.) Environmental Protection Agency (EPA) recognized that many new or expanded facilities that had been constructed in the 1970’s with federal funding assistance were not performing as intended (EPA, 1979; EPA, 1980). To address this issue, the U.S. supported the development of the Composite Correction Program (CCP) as a means of evaluating STPs to determine the underlying cause(s) of poor performance (EPA, 1984; EPA, 1985). In several U.S. jurisdictions, an STP owner was required to undertake a CCP if the STP was not in compliance with regulatory requirements.

In Canada, at about the same time, Environment Canada’s Wastewater Technology Centre (WTC) developed the Process Audit as a comprehensive performance evaluation and energy conservation tool (Speirs and Stephenson, 1985). This optimization approach was not specifically driven by poor performance; rather, it was seen as an effective means of evaluating the capacity of an existing sewage works. As a result, some government capital works assistance programs gave consideration for funding to sewage works that had been subject to an optimization program such as a Process Audit.

Over time, optimization of sewage works (and other municipal infrastructure) has become more common and, in some instances, has been adopted by municipalities with multiple facilities, both water and sewage treatment plants, as a routine part of their operation (Wilson, 2009; Wheeler, 2009). Optimization as a tool to achieve continuous improvement is now widely accepted; however, the following activities may warrant a more detailed optimization study of a specific sewage works or process:

- recurring non-compliance or poor performance, particularly as mandated by a Provincial Officer's Order(s);
- a need to increase rated capacity due to growth in the service area;
- a requirement or desire to achieve a higher level of treatment in terms of effluent quality; and/or
- a need to reduce operating cost due to escalating cost for energy, chemicals or other operational requirements.

Case histories presented elsewhere in the Guidance Manual document performance improvements as well as operating and capital cost savings that have been realized by the successful optimization of sewage works. Realizing some of these benefits is ample reason to implement an on-going program of sewage works optimization.

1.2.1 Value Engineering and Optimization

Value engineering (VE) is a systematic approach used to evaluate an engineering project with the objective of improving its value. Normally, VEs are undertaken at various stages of a design project to determine if the value of the project can be improved by using alternative design approaches. VEs will typically involve a team of experts with expertise in a variety of relevant engineering disciplines, construction and costing in a multi-day workshop environment. VEs have been shown to successfully reduce project construction costs while ensuring that the basis objectives of the project are preserved.

VEs can add value to optimization projects either at the planning stage or during the project execution by serving as a forum for peer review of the work plan, the results and the recommendations. The Workshops described in Section 21.3 of this Guidance Manual could be conducted using the principles of value engineering and involving a VE facilitator and a team of experts knowledgeable in sewage works design, operation and optimization.

1.3 WHAT ARE THE BENEFITS OF OPTIMIZATION

Optimization of sewage works in Ontario, across Canada and internationally has been shown to deliver benefits to the owner/operator, ranging from capital cost savings during plant expansions, improvements in performance and reliability, to operating cost reductions. Numerous example case histories are presented in this Guidance Manual. Some select examples are summarized briefly below.

It is important to recognize that when a sewage works is optimized to increase capacity or meet more stringent effluent limits than the works were originally designed to achieve, the safety margin that was included in the original design will be reduced. Increased attention to the plant operating conditions may be required to ensure that the optimized works continue to consistently achieve the new requirements.

1.3.1 Reduce the Capital Cost of Expansion or Upgrading

Design guidelines for sewage works are, by necessity, conservative as they are intended to ensure that the works are capable of achieving an appropriate level of performance on a consistent basis by providing a margin of safety in the design, particularly when adequate historic data are lacking. Some of the tools described in the Guidance Manual, such as Stress Tests, can be effectively used to document that a unit process can achieve the required performance level at hydraulic or organic loading higher than typically stated in design guidelines. If such is the case, significant capital cost savings can be realized when the facility is expanded or an expansion could be deferred. In some cases, the facility could be re-rated to a higher rated capacity with no or minimal construction of new works.

- Field studies and process modelling were used successfully at the Region of Halton's Mid-Halton Water Pollution Control Plant (WPCP) to substantiate a successful application for plant re-rating (M. Hribljan, 1995).

- Stress testing of the Region of Durham's Corbett Creek WPCP demonstrated that the rated capacity of the facility could be increased from 72,700 m^3/d to 84,400 m^3/d with minimal capital expenditure (XCG, 2000).

1.3.2 Achieve Stricter Standards

Optimization approaches have been used to demonstrate that new or more stringent effluent quality limits for parameters such as total phosphorus (TP) and total ammonia-nitrogen (TAN) can be achieved at some facilities without costly capital works. This is particularly relevant now as concerns regarding nutrients discharged to the receiving water environment have become more significant.

- As part of the Collingwood Harbour Remedial Action Plan (RAP), the Collingwood WPCP was required to significantly reduce the loading of TP discharged from the works. Optimization of the chemical phosphorus removal process at the plant demonstrated that the more stringent requirement could be met without the need to construct tertiary filters, saving an estimated $6.0M (CH2M Hill, 1991).

- A comprehensive process evaluation and optimization study conducted at the City of Windsor's Little River WPCP demonstrated that this facility was capable of achieving nitrification although this was not a design objective. This was achieved through some physical upgrades and with the implementation of accurate and consistent SRT control, resulting in an increase in plant rating and deferral of an estimated $4.6M expansion (Environment Canada, 2003).

1.3.3 Improve Performance

Improvements in performance through operational improvements or improved process control can often bring a sewage works into compliance with its regulatory requirements or improve the reliability of the works. The EPA's CCP was developed specifically to address plants that were unable to achieve their regulatory requirements (EPA, 1984). This same approach has been widely used in Ontario (Wheeler *et al.*, 1994). There are many successful examples of the utility of this approach in STPs that are shown to have been appropriately designed to produce an acceptable effluent quality.

- The Region of Halton has adopted the CCP as its preferred optimization tool. The application of the CCP at the Region's Burlington Skyway WPCP resulted in a substantial improvement in the plant's phosphorus removal performance, as well as demonstrating that the facility could achieve nitrification without major capital expenditure (Wheeler and Hegg, 1999).

- The Region of Durham's Newcastle WPCP had a history of settleability problems that required frequent re-seeding of the bioreactors from another of the Region's facilities. An optimization program demonstrated that the filamentous organisms responsible for the poor settleability could be controlled by process changes, resulting in a significant improvement in plant operation (Hansler *et al.*, 2006).

1.3.4 Reduce Operating Cost

Optimization can identify opportunities to reduce chemical cost and/or improve energy use efficiency. Energy use reduction in STPs can help to mitigate the factors leading to climate change.

- The Region of Halton reduced chemical use for phosphorus removal at their Burlington Skyway WPCP by about 30 percent as a result of an optimization program, resulting in estimated annual chemical cost savings of about $30,000 (Eastwood and Murphy, 1991).

- Optimization and automation of aeration equipment at the Tillsonburg WPCP resulted in power savings of about 15 percent, with a similar reduction in power cost. Subsequent investigations of on-off aeration at the same facility showed that between 16 and 26 percent of the aeration system energy use could be saved, while at the same time achieving a high level of nitrogen removal through the denitrification process (Phagoo *et al.*, 1996).

1.4 WHAT DOES OPTIMIZATION COST AND HOW LONG DOES IT TAKE

The cost and duration of a sewage works optimization program depend on a number of variables, including:

- The project scope and objectives;
- Plant location, size, complexity and configuration;
- Maintenance and construction activities underway at the facility that affect the availability of unit processes or equipment for testing;
- Type and duration of field investigations;
- Level of support provided by the owner/operator;
- Equipment required to execute the field program;
- Sampling and analytical costs;
- Approval requirements; and
- Reporting requirements.

It should be recognized in considering the time required to complete an optimization program that optimization is an iterative and on-going process that involves continuous review of the performance, cost, capacity, and capability of the works. While a specific optimization project may be completed, further opportunities for optimization of the works may be identified.

Stress testing of biological processes often covers multiple seasons, particularly if an objective of the stress test is to demonstrate whether nitrification can be effectively achieved. Conversely, stress testing of clarifiers or other physical-chemical processes can be conducted in a few days.

The cost of an optimization program can range from about $20,000 to conduct the Comprehensive Performance Evaluation (CPE) phase of the CCP at a small- to medium-sized STP, to about $50,000 for a full CCP including the Comprehensive Technical Assistance (CTA) phase (costs based on 2010 dollars). Stress testing and process audit activities to re-rate a small- or medium-sized STP, including multi-season testing of the biological processes, can range in cost from about $80,000 to $120,000 (costs based on 2010 dollars). A comprehensive performance evaluation of all liquid treatment processes, including clarifier stress testing, dye testing, hydraulic modelling, process modelling, evaluation of flow instrumentation, and other activities at a large STP can cost up to $500,000, inclusive of analytical cost (costs based on 2010 dollars). These cost ranges are a guide to the cost to undertake an optimization program, but should not be used for budgetary purposes. A detailed Terms of Reference should be developed with specific tasks and activities identified and used as the basis for estimating the cost of a proposed optimization program.

As shown by the case histories presented in this Guidance Manual, the cost for optimization are often recovered in the form of reduced capital cost for plant expansions and/or reduced operating cost. There are also often the non-monetary benefits of improved operation, improved performance and enhanced plant reliability.

1.5 WHO SHOULD CONDUCT THE OPTIMIZATION

Optimization of a sewage works must involve active participation of the owner and the operating authority, if different from the owner. The owner should establish the objectives of the optimization program and maintain an involvement throughout the process. Operations staff play a critical role in identifying performance limitations or capacity restrictions in the facility based on their hands-on experience in operating the works. They also can assist with conducting specific testing or sampling during the field test program. This can result in an enhanced level of process knowledge and a better understanding of process control options and outcomes, with a resulting benefit in continued optimization of the works through a continuous improvement program. As described in Chapter 3, operations staff must be involved in the development and implementation of Standard Operating Procedures (SOPs) related to the works that they operate.

Some elements of process optimization are best undertaken by experienced process engineering professionals. Some of the test methods described in the Guidance Manual require specialized equipment and training. In addition, the interpretation of the resulting information often is best accomplished by an experienced sewage treatment process engineer.

It is often prudent to include representatives from the regulator, which in Ontario is the Ministry of the Environment (MOE). This would include representatives of the local MOE office (Regional or District Office), and might also include representatives of the Environmental Assessment and Approvals Branch (EAAB) and Standards Development Branch (SDB). Any approvals necessary to undertake the optimization program should be discussed with EAAB and the local MOE office. Appropriate contingency plans should be in place in the event that there are any unexpected short term impacts on effluent quality during field testing. Pre-consultation with MOE and reference to the newest edition of the ministry document *Guide for Applying for Approval of Municipal and Private Water and Sewage Works* will ensure that the optimization program is sufficient to support any future approval applications.

1.6 WHAT ARE THE GENERAL APPROACHES TO OPTIMIZATION

This section of the Guidance Manual provides a brief introduction to some of the more common approaches used for sewage works optimization. These approaches are not mutually exclusive but rather are complementary and are often used concurrently depending on the program objectives.

More detailed discussions are provided in subsequent chapters as referenced herein.

1.6.1 Operator Training and Management Systems

It is recognized that a well-trained operations staff with process control skills and an understanding of sewage treatment processes can produce a quality effluent from a marginal facility. When supported by a management team that encourages optimization and ensures that adequate resources are available to operations staff, an optimized sewage works is often realized. The development of an empowered operations staff is the focus of the CTA phase of the CCP, which is discussed in detail in Chapter 4.

A Quality Management System (QMS) is a set of policies and procedures that an organization develops and follows to achieve a quality product and ensure customer satisfaction. In the context of sewage works, the quality product is considered to be an effluent or biosolids meeting established quality standards at a sustainable cost. The International Organization for Standardization's ISO 9000 series sets the standards for a QMS, establishing the principles and processes involved in the delivery of a product or service. Organizations can become certified to ISO 9001 to demonstrate their compliance to the standard. The standard includes a requirement for continual improvement.

In Ontario, the MOE, as part of the Municipal Drinking Water Licensing Program, requires the implementation of a QMS, as described by the Drinking Water Quality Management Standard (DWQMS). Some Ontario municipalities have voluntarily broadened the application of the QMS to include other infrastructure, such as their sewage works, with the intention of achieving greater efficiency and effectiveness, and greater accountability in these operations.

Similar to the ISO 9000 series, the ISO 14000 series of international standards is a set of policies and procedures related specifically to Environmental Management Systems (EMS). The aim of this standard is to reduce the environmental footprint of a business and to decrease the amount of pollution or waste that the business generates. As with ISO 9001, a business can become certified to demonstrate their compliance to the standard.

More detailed discussion of the role of Operator Training and Management Systems in optimization of sewage works is provided in Chapter 3 of the Guidance Manual.

1.6.2 Composite Correction Program (CCP)

As noted previously, the CCP was developed by the EPA to identify factors that limit the performance of STPs. The CCP has been demonstrated in Ontario to be an effective tool for assessing and optimizing STPs and an Ontario version of the procedure has been developed for use in sewage treatment plants and water treatment plants (Wastewater Technology Centre and Process Applications Inc., 1994; XCG Consultants Ltd., 1992).

The CCP is a two-step process. The first step, termed the Comprehensive Performance Evaluation (CPE), evaluates the operation, design, maintenance and administration of the sewage treatment plant to determine which factors are affecting plant performance and their relative importance. If the CPE determines that the design of the sewage works should be adequate to allow the performance requirements to be met consistently, then the next step in the CCP process, termed the Comprehensive Technical Assistance (CTA), is initiated.

In the CTA, the performance limiting factors identified in the CPE are addressed with the goal of achieving the desired performance. The emphasis of the CTA is on providing operator assistance with process control to ensure that the performance achieved when the CTA is complete can be maintained by a well-trained operations staff.

More detailed discussion of the role of CCPs in optimization of sewage works is provided in Chapter 4 of the Guidance Manual.

1.6.3 Process Audit

The Process Audit was developed by Environment Canada as a tool for evaluating plant performance, capacity and energy use using evolving on-line instrumentation and microcomputer technology. The Process Audit was demonstrated in the 1980's at the Tillsonburg WPCP (Speirs and Stephenson, 1985) and was then applied at numerous full-scale facilities in Canada and the U.S. The fundamental element of the Process Audit in its early development was the use of real-time data to characterize process operating conditions, although the real-time data collection was supplemented by other more conventional analysis tools such as stress testing, clarifier flow pattern analysis, and a general process evaluation.

In 1996, the Ontario Ministry of Environment and Energy (MOEE), Environment Canada (EC) and the Water Environment Association of Ontario (WEAO) jointly developed the *Guidance Manual for Sewage Treatment Plant Process Audits* (MOEE *et al*., 1996) to document the process audit approach. As described in *Design Guidelines for Sewage Works* (MOE, 2008), a process audit is regarded by MOE as being a minimum requirement to support a proposed re-rating of a sewage works to a higher capacity where no new works are constructed.

As on-line instrumentation and Supervisory Control and Data Acquisition (SCADA) systems became more prevalent and more reliable, the installation of temporary instrumentation and data acquisition equipment in full-scale STPs to collect real-time data became less critical to the evaluation of process performance because adequate real-time, dynamic data is often available from the sewage works' SCADA system to support the optimization study. In many Process Audits or optimization studies undertaken since the 1990's, real-time data for key parameters, such as flow and dissolved oxygen, has been acquired from the plant SCADA system to support the plant evaluation. There can still be benefits associated with the installation of more sophisticated instrumentation to measure such parameters as total suspended solids (TSS) or TAN. Further, it must be recognized that all instrumentation used for optimization or routine monitoring and control must be properly calibrated and maintained. Conventional sampling and monitoring approaches are often incapable of detecting the dynamic effects of plant operation on plant performance.

More detailed discussion of the role of the Process Audit in optimization of sewage works is provided in Chapter 5 of the Guidance Manual. The *Guidance Manual for Sewage Treatment Plant Process Audits* (MOEE *et al*., 1996) should be referenced for more detailed information regarding process audits.

1.6.4 Modelling and Simulation

Numerical models can be used as tools to support the assessment of plant performance and capacity as well as a means of predicting the impact of design or process changes on performance and capacity. There are several areas where modelling and simulation can be used to support sewage treatment plant optimization.

- Biological process models can be used to estimate the capacity of a biological treatment process and the ability to achieve more stringent effluent limits (e.g. nitrification or nitrogen removal) without major capital expansion. These models can also be used to evaluate process changes, minor reactor modifications (i.e. - selectors) and system upgrades (i.e. - aeration retrofits).

- Hydraulic models of the sewage treatment plant or sewage collection system can be used to identify hydraulic bottlenecks in the sewage works that may limit the ability to treat peak flows without bypassing.

- Clarifier models can be used to estimate the effects of baffling or other clarifier modifications on clarifier performance or capacity.

- Mixing models, such as Computational Fluid Dynamics (CFD) models, can be used to assess the degree of short-circuiting or dead-space in chlorine contact tanks, digesters or other reactors.

More detailed discussion of the role of modelling and simulation in optimization of sewage works is provided in Chapter 6 of the Guidance Manual.

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CHAPTER 2

OBJECTIVES OF THE GUIDANCE MANUAL

CHAPTER 2

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2.1 PURPOSE OF THE MANUAL

This Guidance Manual is intended for sewage works owners, managers, designers, process engineers, and operators who have an interest in improving the operation and/or performance of a sewage works, reducing the operating costs, or minimizing the capital cost of upgrading or expanding. Users should have a sound understanding of sewage works process design fundamentals and sewage collection systems as these are not covered in this Manual. Other references (Metcalf & Eddy, 2003; WEF/ASCE, 1998; WEF/ASCE, 2007) are available to the user that explain in detail the fundamentals of sewage works process design and operation.

The purpose of the Guidance Manual is to provide those with an interest in sewage works optimization with a source book that describes specific monitoring, testing, and optimization approaches that can be used to evaluate and optimize sewage works.

2.2 USING THE GUIDANCE MANUAL

This Guidance Manual provides a description of optimization approaches that could be applied to all components of sewage works, namely: the sewage collection system, the liquid treatment process train, and the solids treatment process train. In this regard, this Guidance Manual recognizes that all parts of the system must be optimized before the performance, capacity and capability of the works can be considered to be fully optimized. It is also important to recognize that optimization of one component of the sewage works may impact the performance of other components. As such, the implications of optimization steps on other unit processes must be considered.

This Guidance Manual provides an overview of some of the general approaches to sewage works optimization, including Operator Training and Management System (Chapter 3), the CCP approach (Chapter 4), the Process Audit (Chapter 5) and the use of modelling and simulation (Chapter 6).

In subsequent chapters (Chapters 7 to 20), optimization approaches that could be applied to individual unit processes are described and discussed. Generally, each chapter describes the purpose and typical performance of the unit process, provides a summary of some of the typical design or operational problems that may be encountered, and describes techniques that could be used to diagnose the cause of poor performance, improve performance, increase capacity, or reduce costs. It is noted that a separate ministry report is being developed that focuses specifically on water and energy conservation for sewage works.

Each of the chapters can be used independently or with other chapters depending on the scope of a sewage works optimization program. If the objective is to troubleshoot or optimize a specific unit process within the sewage works, then reference should be made to the contents of the chapter dealing with that unit process. If a works-wide optimization program is undertaken, reference should be made to the overview chapters and to unit process chapters that are relevant to the particular works being optimized. In all cases, the references included in each chapter should be reviewed to provide additional information about specific test procedures. As noted previously, the impact of optimizing one unit process on other unit processes needs to be considered in the planning and execution of an optimization program.

The critical first step in sewage works optimization is the development of a comprehensive scope of work or terms of reference. The historic data analysis (Section 5.2.2) or CPE (Section 4.2) should be used to prioritize the work to be undertaken. Subsequently, specific unit processes that limit performance or capacity can be tested and optimized using the procedures described in this Guidance Manual.

Prior to some field tests, such as stress testing, pilot studies or tracer testing, it may be necessary to notify the public or the regulatory agencies. In addition, there can be health and safety issues related to some testing. These aspects are discussed in Section 5.3, but it is important that the regulator be contacted prior to testing to determine what, if any, approvals are necessary. Trained and experienced technologists and technicians should be involved in undertaking the field tests and a rigorous health and safety plan should be developed prior to testing and followed during test execution.

New or improved optimization techniques are being developed on a regular basis. The relevant published literature should be reviewed regularly to update the information presented in this Guidance Manual.

2.3 OTHER SUPPORTING MANUALS AND REPORTS

This manual refers extensively to and should be used in conjunction with other guidance manuals and reports that have been published by the Ministry of the Environment (MOE), Environment Canada (EC), and the Water Environment Association of Ontario (WEAO), including:

- The *Ontario Composite Correction Program (CCP) Manual* (WTC and Process Applications Inc., 1995);
- The *Comprehensive Performance Evaluation (CPE) Manual* (MOEE, 1994);
- The *Comprehensive Technical Assistance (CTA) Manual* (MOEE and WTC, 1995); and
- The *Guidance Manual for Sewage Treatment Plant Process Audits* (MOEE *et al*., 1996).

This Guidance Manual has utilized information contained in these earlier documents and updated information based on more recent source material.

The *Guidance Manual for Sewage Treatment Plant Process Audits* (MOEE *et al*., 1996) contains considerably more detailed information on the methods used to perform specific optimization tests sych as oxygen transfer testing and clarifier dye testing. The PA Guidance Manual also includes example log sheets and data collection forms that can be used to document the results of various tests. This detailed information is not reproduced in this Guidance Manual. The reader should refer to the PA Guidance Manual for this information.

In addition to the guidance manuals identified above, the Sewage Treatment Plant Self Assessment Report allows a sewage works owner/operator to evaluate the performance and the limitations of a sewage works to determine whether an optimization program might be beneficial. For convenience, the Self Assessment Report is appended to the Guidance Manual as Appendix B.

The Managers Guide to Sewage Treatment Plant Optimization (WTC and Process Applications Inc., 1996) provides sewage treatment plant managers with a comprehensive overview of several optimization methods that can be used to optimize a sewage treatment plant. For convenience, the Managers Guide is appended to this Guidance Manual as Appendix C.

2.4 REFERENCES

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CHAPTER 3

OPERATOR TRAINING AND MANAGEMENT SYSTEMS

CHAPTER 3

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3.1 OPERATOR TRAINING

Optimization of a sewage works should include increasing the capabilities and knowledge of the operations and management staff of the works and improving the performance of the equipment and the treatment processes to be effective and sustainable.

Developing a capable and empowered operations staff with supportive management and appropriate operations and maintenance (O&M) procedures and practices is critical to achieving and maintaining a high level of performance in the sewage works. The success of the CCP approach (Chapter 4) is, to a large extent, due to the transfer of skills and knowledge to the operations and management staff during the CTA phase.

Operator training should not be confused with operator certification or licensing which is regulated in Ontario under O. Reg. 129/04. The objective of the mandatory operator certification program (O. Reg. 129/04) is to ensure that sewage works operators have the necessary education, training, knowledge and experience to operate the works. Certification is based on passing licensing exams and attending 40 hours of professional development or training per year.

CCPs undertaken in Ontario (MOEE and WTC, 1995) and elsewhere (EPA, 1979; EPA 1980), and other studies (XCG, 1992), have consistently shown that the most common performance limiting factors in sewage works are the lack of appropriate process control techniques and the limited application of process control concepts.

Providing operations staff with the knowledge, ability and tools needed to achieve a consistent level of process control at the works should involve a combination of classroom and hands-on training. The classroom training is aimed at explaining the fundamental concepts of sewage treatment and process control. The hands-on training is intended to demonstrate how the concepts apply to the specific works that are being operated. There are numerous sources of classroom training available. Acquiring the requisite hands-on training in monitoring and process control techniques is more difficult and expensive than classroom training, particularly for smaller works that may not have in-house staff capable of providing hands-on training. A regional approach to delivery of hands-on training can mitigate the higher costs of this type of training for small facilities.

3.2 OPERATIONS MANUALS

Certificates of Approval (Cs of A) for Sewage Works commonly require that an Operations Manual be prepared and maintained for the works. The Manual should include:

- Operating procedures for routine operation of the works;
- Inspection programs and the methods or tests employed to determine when maintenance is necessary;
- Repair and maintenance programs;
- Procedures for the inspection and calibration of monitoring equipment;
- A spill prevention control and contingency plan; and
- Procedures for receiving, responding to, and recording public complaints including recording any follow-up actions taken.

The Licensing Guide (MOE, 2004) requires that Operations Manuals be reviewed and updated at least once every two years or as needed to reflect changes in design or operating conditions.

Operations Manuals should include:

- A description of the works;
- A general description of the individual unit processes, including the sewage collection system;
- Standard Operating Procedures (SOPs); and
- Contingency plans to deal with unforeseen situations.

The description of the works should include, as a minimum, basic information such as the location of the works, design flows and loadings, a process flow diagram, number, dimensions and sizes of major tanks or reactors, a process and instrumentation drawing (P&ID) for the works, and the performance and monitoring requirements as specified in the C of A.

Each unit process should be described with basic design information, location within the works, number of units, normal operational ranges for key operating parameters, unit specific P&ID, and any relevant health and safety considerations.

Simple and straightforward SOPs should be included in the Operations Manual. The use of SOPs by all operations staff will help to achieve a consistent operation. The specific SOPs needed for a particular works should be determined by knowledgeable operations and management staff at the works and should consider industry best practices. SOPs should be developed in a format that allows for easy revision as improved operating procedures are identified or new processes or equipment are added. They should follow a consistent format or template, and be clear and concise.

The contingency plans should anticipate unusual or emergency situations. The intention is to provide operations staff with clear and concise information on the steps that should be taken to respond to the specific emergency. Key and mandatory contact information should be included.

Operations staff should be involved in the preparation and updating of the Operations Manual and the preparation of SOPs. The Operations Manual should:

- Document the sampling and testing procedures to be used to define the operating condition of the works or process;
- Provide a summary of the appropriate operating condition; and
- Identify the actions that should be taken in response to the monitoring result if the operating condition is not appropriate. Sample log sheets and calculations should be provided.

Sophisticated electronic Operations Manuals linked to record drawings and Geographical Information Systems (GIS) mapping of the works are possible and are becoming more common, particularly at larger facilities. Whether electronic or paper manuals are used, a simple means of revising and updating the document is important.

3.3 MANAGEMENT SYSTEMS

A Quality Management System (QMS) is a compilation of policies and practices and the supporting infrastructure that an organization uses to reduce or eliminate non-conformance with specifications or standards applicable to its product or service.

Although management systems and standards have been in use for many years, the release in 1987 of the International Organization for Standardization (ISO) 9001 Quality Management System Standard led to a much broader acceptance and implementation of QMS. In Ontario, establishing and maintaining a QMS is a mandated requirement for owners and operating authorities of municipal residential drinking-water systems based on the requirements of the Drinking Water Quality Management Standard (DWQMS), (MOE, 2007).

QMS is equally applicable to sewage works and some utilities and municipalities have expanded the concept of DWQMS to cover their sewage works (McCormick, 2009). The QMS formalizes the management and operational procedures used at the works. It sets specific objectives, specifies the procedures to be used to meet those objectives, identifies the methods or metrics to be used to measure the effectiveness of the actions, and emphasizes the need for continuous improvement through a cycle of action and review.

The continuous improvement component of QMS is directly related to optimization and QMS offers a means of tracking and monitoring the improvement achieved. Guidance is available from MOE (MOE, 2007) on the implementation of QMS in a drinking water system that could be used to develop and implement an equivalent system for a sewage works.

ISO has also developed a series of standards and guidelines for an Environmental Management System (EMS), the ISO 14000 series. A business or utility can use these standards or guidelines to set performance targets and to establish a monitoring framework to assess compliance with the standards and compare actual performance to the targets. In the sewage industry in the U.S. and Canada, EMS has been most commonly applied to biosolids land application programs. MOE has supported a demonstration of the application and benefit of EMS to biosolids (CH2M Hill and PA Consulting Group, 2002). Some water and sewage works owners and operators, including the Region of York and the Region of Waterloo, have had their water and/or sewage works certified to the ISO 14001 standard with the goal of improving performance and compliance.

3.4 REFERENCES

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CHAPTER 4

COMPOSITE CORRECTION PROGRAM (CCP)

CHAPTER 4

COMPOSITE CORRECTION PROGRAM (CCP)

4.1 BACKGROUND

The Composite Correction Program (CCP) was developed by the EPA to identify and mitigate problems of poor performance in STPs. Several manuals are available describing the rationale for and the procedures involved in the CCP (EPA, 1979; EPA, 1980; EPA, 1984; EPA, 1990). These reports and guidance documents present the detailed, step-by-step approach involved in the CCP.

Similarly, the CCP was demonstrated in Ontario in the early 1990's (MOEE, 1995; MOEE, 1994), and an Ontario Guidance Manual (WTC and Process Applications Inc., 1996) was prepared that modified the information in the U.S. guidance documents to reflect the design and operation of STPs in Ontario. It is not the intention of this Guidance Manual to reproduce the information contained in the earlier CCP-specific manuals. Those interested in applying the CCP approach to optimize STPs should refer to the detailed information contained in the referenced material. This Chapter of the Optimization Guidance Manual for Sewage Works will provide an overview of the CCP and provide brief case histories demonstrating the performance improvements that have been achieved using this program.

As described in Section 1.6.2, the CCP comprises two steps, the Comprehensive Performance Evaluation (CPE) and the Comprehensive Technical Assistance (CTA). The relationship between the CPE and the CTA is illustrated in Figure 4-1. Each component of the CCP is described briefly below.

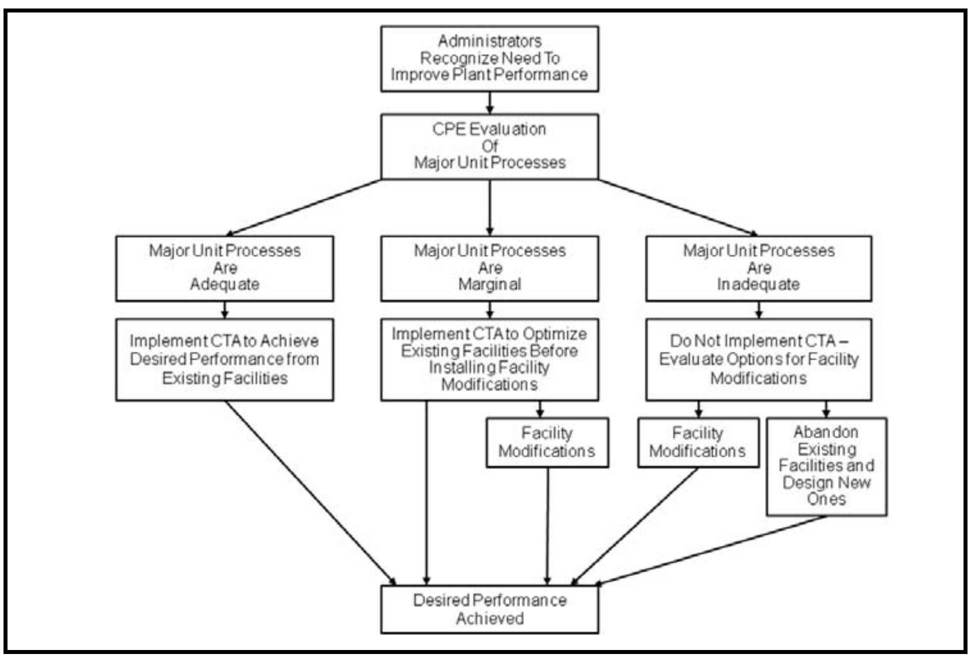

Figure 4-1 - Overview of Composite Correction Program

(Adapted from EPA, 1990)

4.2 COMPREHENSIVE PERFORMANCE EVALUATION (CPE)

The first step of the CCP, the CPE, is intended to determine if the STP has the capability of treating the sewage being received to the levels required by the C of A. The design, operation, maintenance, and administration of the STP are assessed against benchmarks contained in the Guidance Manuals cited in Section 4.1 and referenced in Section 4.5 and performance limiting factors, if any, are identified.

If the CPE determines that the facility should be capable of achieving the required performance, then the second step of the CCP, the CTA, is initiated to address the identified performance limiting factors through the transfer of improved operational and process control techniques to the operations staff or minor upgrades to monitoring, metering, or process equipment.

If the CPE identifies that the facility is not capable of achieving the required performance, then design upgrades or further studies need to be undertaken to produce a facility that is considered to be capable of the required level of performance.

The CPE consists of the following steps conducted by an experienced CPE team knowledgeable in the design, operation, maintenance and administration of sewage works:

- A kickoff meeting, involving operations staff, management staff and the CPE team at which the purpose of the CPE process, and the CPE schedule are explained;

- A plant tour, led by a senior plant operator, to familiarize the CPE team with the facility and to obtain information on maintenance and operational practices;

- An assessment of performance based on historic data and including a Sludge Accountability Analysis (Section 5.2.2) to assess the validity of the historic data;

- An evaluation of the major unit processes against benchmarks to determine if they are operating within generally accepted design conditions;

- Plant personnel interviews with key staff to obtain information on how the facility is operated, maintained and managed;

- Determination of performance limiting factors based on a review and analysis of all of the information collected during the above-noted activities;

- An exit meeting to present the findings, including the performance limiting factors identified; and

- A CPE report prepared to summarize the findings that were presented at the exit meeting.

The on-site component of the CPE, excluding the preparation of the CPE report which is done after the CPE team leaves the site, normally involves about one week of intensive work by two to three evaluators. The CCP process is best suited to the evaluation of small- to medium-sized STPs that can be effectively evaluated by the CPE team during a one-week period on-site.

The key element of the CPE is the identification of performance limiting factors. The CPE Guidance Manuals list 70 potential factors that could limit performance in four broad categories: Administration, Design, Operations, and Maintenance. After the factors that limit performance are identified, each is ranked according to its effect on plant performance according to the classification system presented in Table 4-1.

Table 4-1 - Classification System for Performance Limiting Factors (EPA, 1984)

Rating	Adverse Effect of Factor on Plant Performance
A	Major effect on a long term repetitive basis.
B	Minimum effect on routine or major effect on a periodic basis.
C	Minor effect.

The CPE rates the STP as Type 1, Type 2 or Type 3 according to its ability to achieve the performance required by its C of A where each type of STP is defined below (EPA, 1984; MOEE, 1995).

Type 1: The existing major unit processes are adequate to meet current treatment requirements.

Type 2: The existing unit processes are marginal but improved performance is likely through the use of a CTA. For Type 2 plants, the CTA focuses on clearly defining the capability of the existing facilities through optimum operations and application of concepts. Individual unit process deficiencies are identified so that modifications can be implemented where required.

Type 3: Major construction is indicated if the plant, as currently designed, is not considered to be capable of meeting current treatment requirements. Typically, a CTA is not implemented at a Type 3 plant until modifications have been completed and a capable (Type 1 or Type 2) plant is available.

As originally conceived, the CCP was intended to take a capable plant that was not meeting its performance requirements and, through upgrading of operating and process control skills during a CTA, produce a plant that is in compliance. In Ontario, the CCP has become a broader based optimization technique that is applied to STPs that are achieving their compliance requirements with the objective of improving specific areas of the operation. This is demonstrated in the case histories presented in Section 4.4.

4.3 COMPREHENSIVE TECHNICAL ASSISTANCE (CTA)

As described above, a CTA is implemented in a Type 1 or Type 2 STP in order to improve performance by addressing the performance limiting factors identified during the CPE.

A CTA involves the systematic training of operations and management staff responsible for the STP so that process control and operating procedures are appropriately and consistently applied and the desired level of performance is achieved. The CTA typically involves a long-term involvement of experienced process engineers, technologists or operators over at least one year to ensure that the appropriate skills are transferred to the plant staff. The long-term involvement ensures that improvements become evident even in processes that have long response times, such as biological treatment processes. It also ensures that seasonal impacts or other factors affecting plant performance are experienced by the operations staff.

A CTA can involve telephone calls to provide direction to plant staff, site visits to provide on-site training or assist with specific testing at the plant, written status reports to document changes made and results achieved, and a final report summarizing the outcomes. A CTA normally involves more intensive site visits and telephone support during the early stages, with a reduced level of involvement over time as the skills are transferred to the operations staff. During a CTA, monitoring equipment may need to be obtained to allow staff to conduct specific process control tests. Process and operations log sheets may need to be developed and SOPs documented. The intention of the CTA is to empower the operations staff to undertake, with guidance, the development of appropriate tools to ensure that adequate process control monitoring and adjustment is conducted.

Unlike a CPE which follows a relatively well-defined protocol, the tasks undertaken in and the duration of a CTA can vary widely depending on the skills and attitudes of the plant operations and management staff. A key to maintaining momentum during a CTA is to provide regular updates that show the process changes that have been implemented by operations staff and the resulting improvements in performance that have been achieved.

4.4 CASE HISTORIES

Detailed examples of CPEs and CTAs conducted in Ontario during the development and demonstration of the CCP are contained in the reference reports (MOEE, 1995; MOEE, 1994). The following two case histories are examples of CCPs undertaken by the Department of National Defence (DND) at their sewage treatment works across Canada (Spätling *et al.*, 2000) and by the Regional Municipality of Halton (Wheeler *et al.*, 1999). A third case history documents the application of CTA by the City of Guelph (Wheeler, 2009).

4.4.1 Sewage Treatment Plant Optimization Program (STPOP)

The following case history is based on information provided in Spätling *et al.* (2000).

Description

In 1995, the DND initiated a Sewage Treatment Plant Optimization Program (STPOP) with the intention of optimizing the performance of seven existing STPs across Canada. The DND STPOP mission was “to promote environmental protection through skills transfer as measured by improved and compliant effluent quality at least cost”.

Approach

A CPE was undertaken in order to evaluate each plant’s design, operation, maintenance and administration to determine whether a CTA was appropriate. The CPE consisted of plant tours, data collection and verification, the evaluation of unit process capacities and identification of critical unit processes, and the identification, classification and ranking of factors limiting performance.

A CTA was conducted initially at plants where the results of the CPE indicated that the plant did not meet effluent limits, and was not the result of major design limitations. CTAs were subsequently conducted at all of the remaining STPs. The CTA process was carried out over the duration of 9 to 24 months for the following reasons:

- To ensure the effectiveness of repetitive training;
- To implement minor design upgrades and administrative changes; and
- To review operating procedures under varying weather conditions throughout the course of all seasons.

The most common performance improvement activities to result from the CTA included:

- Installation of automatic composite samplers in winterized enclosures;
- Conducting workshops and providing training in the assessment and control of solids within the system;
- Identification of trends from process and performance charts, and enhancement of operations staff interpretation skills; and
- Conduct special studies to evaluate and optimize individual unit processes (e.g. removal efficiency of shallow primary clarifiers during wet weather flow events).

Following completion of the CTA phase, a performance maintenance phase was initiated to maintain the improved performance, promote accountability for plant performance, and to disseminate knowledge and experience between STPs. Monthly reviews and annual on-site performance evaluations such as sludge accountability analyses were conducted to evaluate the plant status against CTA recommendations.

Results

Table 4-2 presents the seven STPs included in the study and the results of the respective CPE and CTA.

Out of the seven STPs evaluated, 8 Wing Trenton and CFB Borden were identified as being unable to achieve sufficient treatment under existing conditions. Nevertheless, CTAs were conducted to assess whether optimized operation and maintenance could lead to performance improvements. Improved process monitoring and procedural changes led to improved operational control. In conjunction with minor plant upgrades, the above changes resulted in improved effluent quality and deferral of an estimated \$2.8M and \$8.0M in capital upgrade expenditure at the 8 Wing Trenton and CFB Borden STPs, respectively. An additional savings of \$5,000 and \$3,000 in annual O&M costs were estimated based on optimization of unit processes and operations at 8 Wing Trenton and CFB Borden, respectively. Figure 4-2 presents an example of the improved effluent quality realized as a result of the CCP at 8 Wing Trenton.

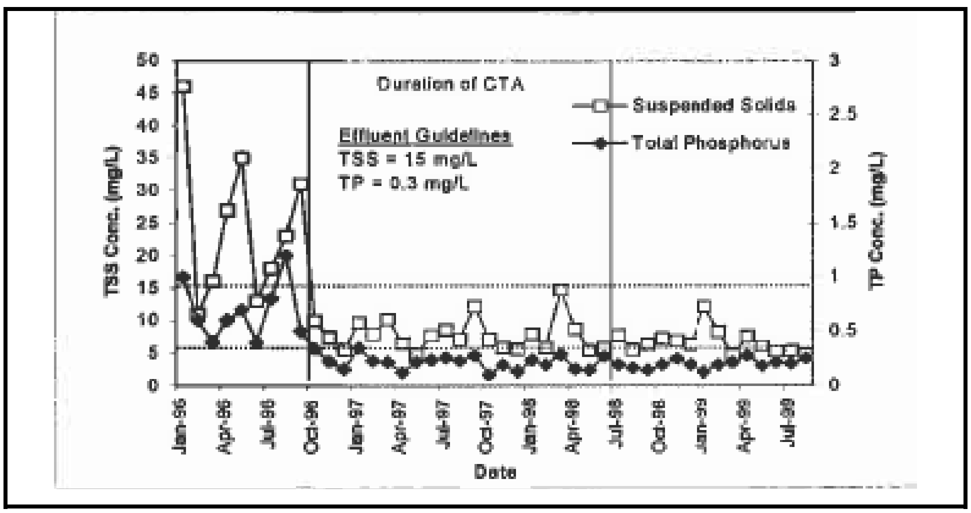

Figure 4-2 - Monthly Average TSS and TP Concentrations at 8 Wing Trenton

(From Spätling *et al.*, 2000)

Table 4-2 - Summary of CPE and CTA Results
(From Spätling *et al.*, 2000)

Base / Province	Process	Design ADF (m^3/d)	A – Rated Performance Limiting Factors	CTA Results
Combat Training Centre Gagetown / New Brunswick	Conventional Activated Sludge	13,600	• Performance Monitoring • Familiarity with Needs	• Improved effluent TP quality
Canadian Forces Base Valcartier / Quebec	Conventional Activated Sludge	5,400	• Familiarity with Needs • Phosphorus Removal Equipment	• Improved effluent TP quality • \$100K annual savings in sludge dewatering
8 Wing Trenton / Ontario	Conventional Activated Sludge	3,400	• Performance Monitoring • Sludge Storage & Disposal • Application of Concepts	• Improved effluent TSS, BOD_5 and TP quality • \$2.8M capital savings • \$5K annual O&M savings
Canadian Forces Base Borden / Ontario	Conventional Activated Sludge	3,785	• Secondary Clarifier • Application of Concepts • Familiarity with Needs	• Improved effluent TSS, BOD_5 and TP quality • \$8.0M capital savings • \$3K annual O & M savings
Area Training Centre Meaford / Ontario	Sutton Process w/ Tertiary Filtration	1,000	• Familiarity with Needs • Performance & Process Monitoring • Advanced Waste Treatment • Application of Concepts	• N/A
17 Wing Winnipeg / Manitoba	Trickling Filter	1,400	• Familiarity with Needs • Performance Monitoring	• Improved effluent TP quality • \$1.2M capital savings • \$250K annual sewer surcharges avoided
4 Wing Cold Lake / Alberta	Oxidation Ditch	4,500	• Familiarity with Needs • Application of Concepts & Testing • Phosphorus Removal Equipment	• N/A

Notes:
N/A – information not available

The following were the three most common limiting factors identified at the STPs:

- Familiarity with Plant Needs - It was determined that plant managers often lacked first hand knowledge of the operational requirements of the STP or a clear understanding of the effluent objectives;
- Performance Monitoring - The use of grab sampling and the results from the CPE did not support the reported plant performance; and
- Application of Concepts and Testing for Process Control - The results from extensive sampling and analysis programs were not effectively utilized as the basis for process control or determining plant needs.

The application of the CCP was considered successful for the DND STPOP. The CCP provided a structured procedure to systematically identify and eliminate deficiencies in design, operation, maintenance and administration. Clear definition of effluent objectives was crucial to developing the necessary focus for optimization activities. The success of the CTA phase was contingent on the support of plant staff. Several challenges encountered in achieving and maintaining optimized plant performance were identified, including aging infrastructure, staff reduction due to departmental reorganization, limited financial resources, and the lack of continuity due to the routine rotation of military staff. It was suggested that data reporting policy should be changed to address and support performance maintenance.

4.4.2 Regional Municipality of Halton Optimization Program Using the CCP Approach

The following case history is based on information provided in Wheeler *et al.* (1999).

Description

In 1995, the Regional Municipality of Halton (Halton Region) implemented a formal optimization program for their sewage treatment facilities. There are seven STPs in Halton Region, the largest of which is the Burlington Skyway Wastewater Treatment Plant (WWTP), which is dealt with specifically in this case history.

Stringent effluent limits applied to the Skyway WWTP by the Hamilton Harbour Remedial Action Plan (HHRAP) facilitated the creation of a formal optimization program for Halton Region. The goals of the formal optimization program were to maximize the hydraulic capacity of the existing infrastructure while improving process performance, and to empower staff with skills and initiative to implement activities to economically maintain the targeted performance levels. The Burlington Skyway WWTP is a conventional activated sludge (CAS) plant with two stage anaerobic digestion and a C of A rated capacity (average daily flow) of 93,000 m^3/d. It was the first of the Halton Region facilities to undergo optimization.

Approach

The optimization program applied to the Skyway WWTP utilized the CCP. For the initial phase of the CCP, a CPE was performed to identify the areas of design, operation, maintenance and administration contributing to suboptimal performance. CTA was then applied to remedy any process limiting factors identified in the CPE.

The performance improvement activities identified in the CTA include:

- Minor process modifications to facilitate increased process control;
- Operational changes for improved process control;
- Improved process understanding and problem solving skills; and
- A pilot study on an alternative sludge collection system to improve process efficiency and control.

Results

The results of the CPE and CTA for the Burlington Skyway WWTP are presented in Table 4-3.

Table 4-3 - Summary of Burlington Skyway WWTP CCP Results

Plant	Identified Performance Limiting Factors	CCP Results
Skyway WWTP	• Secondary Clarifier Flow Split • Sludge Removal Equipment • Polymer Dosage Equipment • Familiarity with Needs • Application of Concepts	• Improved effluent quality • Achieved HHRAP seasonal TAN target of 5.6 mg/L in cold weather • $33M capital savings • Deferral of $17M in expansion upgrades

Following the CPE, the following design modifications were implemented to correct physical limitations inhibiting performance:

- Installation of flow splitting device to balance the flows to the secondary clarifiers - This corrected the uneven loading to the secondary clarifiers;
- Installation of dedicated chemical feed pumps for each bank of secondary clarifiers - Dedicated chemical feed pumps allow greater control and flexibility of the phosphorus removal system; and

- Retrofit of existing secondary clarifier sludge removal scrapers – Improved the efficiency of the sludge removal mechanism, minimizing the sludge mass in the secondary clarifiers while maintaining the targeted return activated sludge (RAS) concentrations.

Concurrent with the design modifications were changes to operational procedures and improvements in plant staff knowledge and skill. The approach used at the Skyway WWTP encouraged technical skill improvement and empowered staff with priority setting and problem solving skills necessary to sustain the improvements realized.

- Familiarity with Plant Needs - Priority setting helped to foster and develop the knowledge of the unique process requirements for the plant, allowing improved application of concepts to process control.

- Application of Concepts - Applying the knowledge gained throughout the CTA process has allowed the Skyway WWTP to achieve and to maintain the effluent targets set out in the HHRAP. It also allowed the operations staff to be able to identify areas where further improvements may be realized.

The plant upgrades, combined with process and procedural changes, led to improved operational control and improved effluent quality. The demonstrated ability to achieve and sustain the HHRAP effluent targets have resulted in capital cost savings of $33M and deferred of an additional $17M in plant expansion upgrades. Achieving consistent plant performance via measured effluent quality made it possible to identify the effects of specific industrial contributors (during a shutdown of a major industry). The observed improvement in system performance prompted the requirement for pretreatment of effluent discharged from the industry, reducing organic loading to the STP and "recovering" some plant capacity. Figure 4-3 presents the average effluent TP loading from the Burlington Skyway WWTP.

The application of the CCP was considered successful at the Skyway WWTP. The CCP provided a structured procedure to systematically identify and eliminate deficiencies in design, operation, maintenance and administration while fostering an environment for improving technical and problem solving skills in operations staff. The result of the CCP was effluent quality meeting the targets outlined in the HHRAP. Halton Region was recognized by the Bay Area Restoration Council for the effects of the Region's efforts on ambient water quality in Hamilton Harbour. Subsequently, the Region has pursued optimization of its other STPs prior to construction of anticipated upgrades.

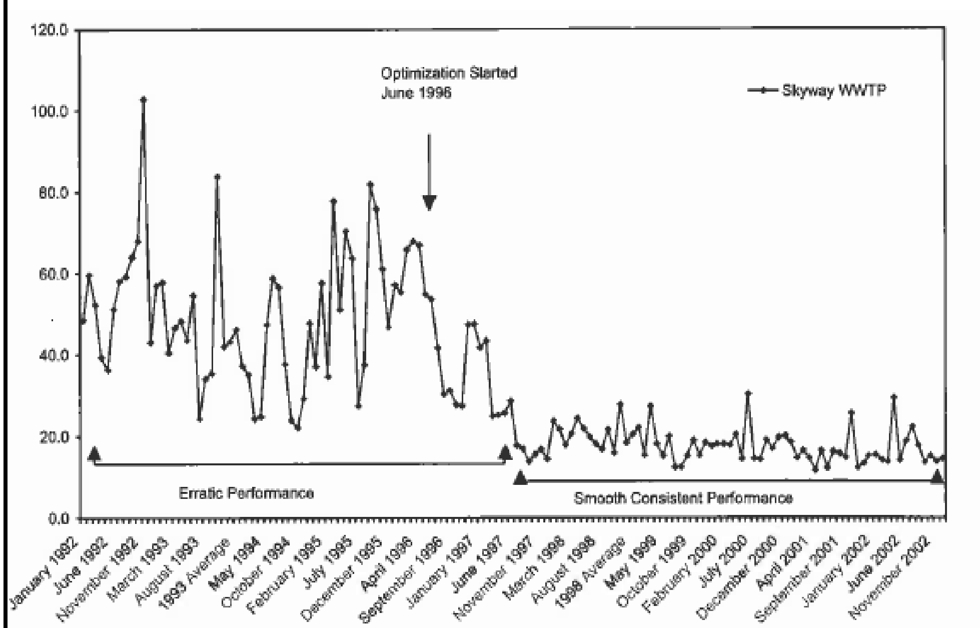

Figure 4-3 - Average Effluent TP Loadings from the Burlington Skyway WWTP

(From Federation of Canadian Municipalities and National Research Council, 2003)

4.4.3 Optimization of the Guelph WWTP Using the CCP Approach

The following case history is based on information provided in Wheeler (2009).

Description

The City of Guelph WWTP is a tertiary treatment facility comprised of four secondary plants with a total C of A rated capacity (average daily flow) of 64,000 m^3/d. The Guelph WWTP discharges to the Speed River, which is a Policy 2 receiving stream. The plant is currently operating at 55,000 m^3/d. As part of a sewage master plan, the construction of a new facility was identified to accommodate growth beyond 64,000 m^3/d.

In 2007, the City of Guelph initiated a comprehensive optimization program for the Guelph WWTP to maximize the performance and capacity of the existing facility. The objectives of the optimization program were the following:

- To improve the nitrification efficiency of the plant to meet the proposed future effluent ammonia limits;
- To maximize the capacity of the existing infrastructure; and
- To document additional capital cost savings realized as a result of the optimization program.

Approach

The plant meets its existing C of A effluent limits. However, the Speed River is a Policy 2 receiving stream, and as such, the proposed future effluent limits are more stringent than the existing limits. The nitrification process in the existing facility was identified as the performance limiting factor, and CTA was applied to improve the system performance.

Operations staff skills, knowledge and understanding of biological activated sludge process control were improved. With the proper support from management, the improved knowledge and skills could be applied to the process, in this case, nitrification, to effect and maintain improvements.

Results

The results of the optimization of the nitrification process at the Guelph WWTP are presented in Table 4-4.

Table 4-4 – Summary of Guelph WWTP Optimization Results

Identified Performance Limiting Factors	Optimization Results
• Poor Process Control • Inadequate Aeration Equipment Operation • Application of Concepts and Testing	• Improved nitrification • Effluent TAN was reduced to below 1.5 mg/L and maintained below the future proposed ammonia limit • $20 M capital savings • Deferral of $11 M in expansion upgrades

The following steps were implemented to correct the identified performance limiting factors:

- Re-prioritizing regular in-house duties to put more emphasis on process control related activities. Priority setting helped to foster and develop the knowledge of the unique process requirements for the plant, allowing improved application of concepts for process control;

- Adjustment of the aeration system blower operation increased the oxygenation in the aeration tanks, improving nitrification performance; and

- Applying the knowledge gained has allowed the Guelph WWTP to achieve the proposed future effluent limits, and allowed the operations staff to be able to identify areas where further improvements may be gained.

The approach used at the Guelph WWTP encouraged technical skills improvement and empowered staff with priority setting and problem solving skills necessary to sustain the improvements realized.

The optimization was initially implemented at one of the four separate plants to identify the potential to meet the proposed future effluent limits. Figure 4-4 presents the secondary effluent ammonia concentrations from the initial optimization of the single plant.

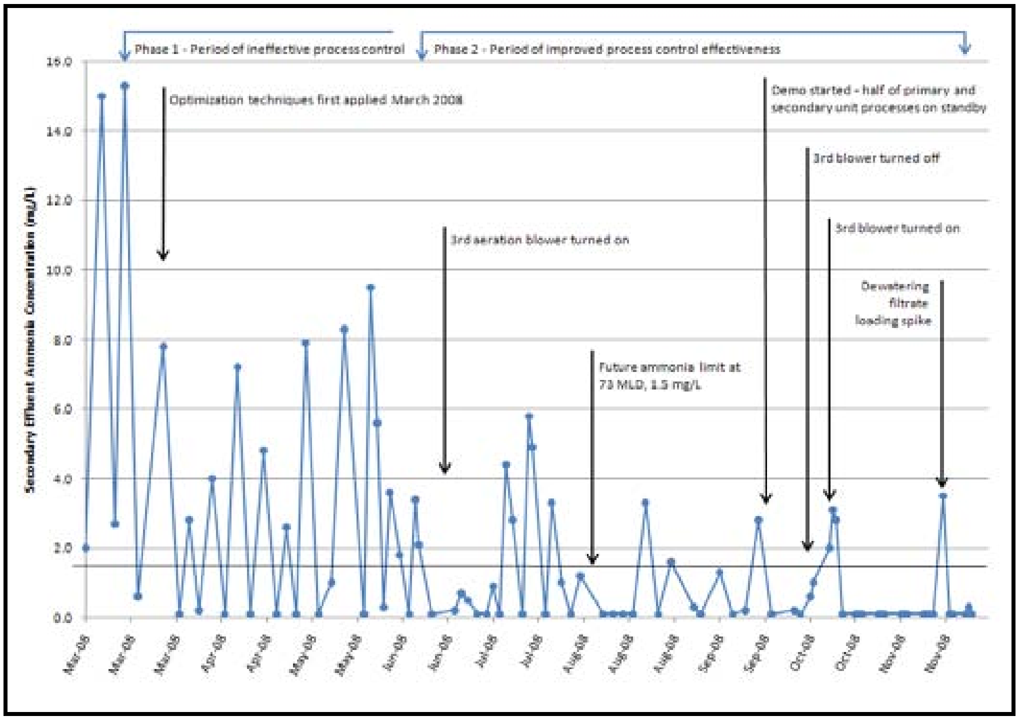

Figure 4-4 - Average Secondary Effluent Ammonia Concentrations from Plant 2

(Adapted from Wheeler, 2009)

Once it was determined that it was feasible for the existing plant to produce effluent in compliance with the proposed future effluent limits, the experience gained at the first plant was applied to the remaining three plants for a full-scale capacity demonstration. Once the changes were implemented and the process stabilized, two of the plants were taken offline, and the full facility flow was treated by the two online plants, demonstrating the increased capacity of each plant as a result of optimization. The optimization knowledge and techniques gained were then applied to the disinfection system and anaerobic digesters, resulting in deferred capital and expansion costs.

The process and procedural changes led to increased operational control and improved nitrification performance, and a demonstrated increase in plant capacity to 80,000 m^3/d. If the results of the optimization remain consistent for the duration of the demonstration, the plant will be re-rated to 80,000 m^3/d. The demonstrated ability to achieve and sustain the proposed future effluent limits, and the potential plant capacity re-rating may result in capital cost savings of $20M. The application of optimization techniques to the disinfection system and anaerobic digesters may result in the deferral of an additional $11M in capital costs and plant expansion upgrades.

The application of CTA provided the Guelph WWTP operations staff with improved technical, process control and problem solving skills. These improved skills resulted in improved nitrification performance. The potential capacity re-rating of the Guelph WWTP may result in an overall savings of $31 M for the city. The positive results have prompted the Grand River Conservation Authority to sponsor the application of the same techniques to the remaining 27 treatment plants in the Grand River Watershed.

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CHAPTER 5

OVERVIEW OF PROCESS AUDIT APPROACH

CHAPTER 5

OVERVIEW OF PROCESS AUDIT APPROACH

5.1 INTRODUCTION TO PROCESS AUDITING

As described in Section 1.6.3, the Process Audit was originally developed as a sewage treatment plant evaluation tool that made extensive use of temporary on-line instrumentation equipment and computers as a means of collecting dynamic information on process operation and performance. STP optimization now commonly depends on the on-line instrumentation and data acquisition equipment permanently installed in the STP, such as supervisory control and data acquisition (SCADA) systems, for data collection; however, the overall concept and approach associated with the Process Audit has become the basic model for sewage works optimization programs, and is cited in Cs of A and the MOE Design Guidelines (MOE, 2008) as an essential component of the documentation and testing needed to support an application to re-rate an STP. The municipality should consult with MOE to determine the study requirements if an objective of the optimization program is to support an application to re-rate the works.

The Process Audit typically involves the following basic steps:

- Establish project objectives;
- Undertake a historic data review and analysis;
- Establish, based on the historic data review, the capacity of the individual unit processes and of the overall works and identify any performance or capacity limitations;
- Develop a field monitoring and testing program to confirm the findings;
- Develop recommendations to mitigate performance or capacity limitations; and
- Report the findings.

These steps are described further in the following subsections. These basic steps should be followed in any sewage works optimization study, whether the project is called a Process Audit or not. Figure 5-1 illustrates schematically the key steps in a sewage works optimization project.

The *Guidance Manual for Sewage Treatment Plant Process Audits* (MOEE *et al*., 1996) provides considerable detail regarding the process audit approach for sewage treatment plant optimization. It includes detailed descriptions of many of the test procedures discussed in this manual and provides data sheets and other supporting information. The reader should refer to the *Guidance Manual for Sewage Treatment Plant Process Audits* for additional information and use the two guidance manuals as companion documents.

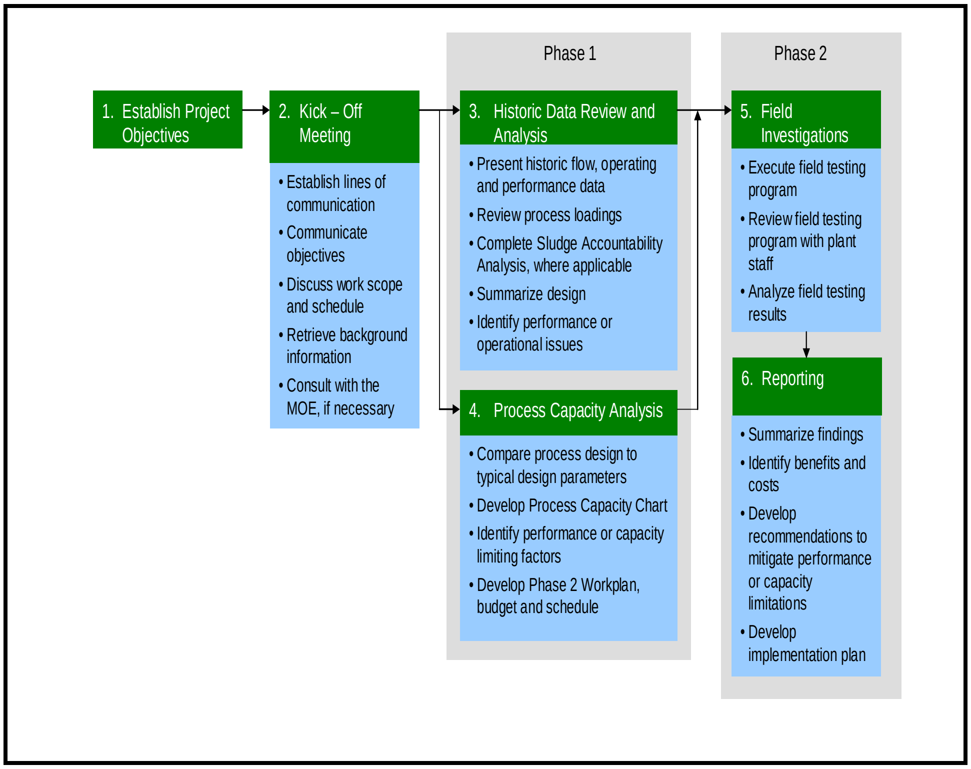

Figure 5-1 - Schematic Illustration of a Sewage Works Optimization Work Plan

5.2 PROCESS AUDIT STEPS

5.2.1 Establish Project Objectives

The objectives of the Process Audit or sewage works optimization program need to be clearly defined at the beginning of the project and communicated to the participants. Those components of the works that will be investigated should be specified. For example, the optimization study could include the entire sewage works (collection system, pumping stations, liquid and sludge treatment processes), only the liquid train treatment processes or the sludge treatment processes, or a specific unit process or unit processes (e.g. aeration system, secondary clarification, anaerobic digestion).

The goal(s) of the Process Audit or sewage works optimization should also be defined and may include, but not necessarily be limited to, any or all of the following:

- Improving performance;
- Increasing capacity;
- Reducing chemical or energy cost;
- Improving the knowledge level or capability of the operations staff; or

- Achieving higher quality effluent in terms of specific parameters (e.g. TAN, TP, TSS).

Only after the objectives are clearly defined is it possible to develop a realistic work plan, budget and schedule for the project.

5.2.2 Historic Data Review

This is generally considered to be Phase 1 of a Process Audit or sewage works optimization program as it will define what, if any, field investigations are needed during the subsequent phase of the project. The scope of the historic data review should be consistent with the project objectives and should focus on those components of the works that are the subject of the optimization study and other components of the works that may be impacted as a result.

Phase 1 should be initiated with a project mobilization or kick-off meeting involving key project staff, including the works owner, operations staff, and members of the consulting team if applicable. The objectives of this meeting are:

- To communicate the project objectives;
- To establish a communication protocol;
- To discuss the project work scope and schedule as understood at this stage of the project; and
- To retrieve the key information that will be required to complete Phase 1 of the project.

Ideally, a listing of Information Needs should be developed prior to the kick-off meeting so that key information can be available at the meeting. Information needs will include such items as:

- Historic operating and performance data (a minimum of three to five years, in electronic format if possible) including flows, sewage characteristics, operational information (chemical use, mixed liquor suspended solids (MLSS) concentrations, dissolved oxygen concentrations, sludge wastage data, etc.), effluent quality data, performance data for intermediary processes, etc.;
- Certificate(s) of Approval for the works;
- Process design or preliminary design reports with information on sizing of key unit processes and mechanical equipment (e.g. blowers, pumps, etc.);
- P&IDs of unit processes;
- Detailed design drawings (as-built or record drawings) of the works;

- Operations manuals;

- Any MOE inspection reports, Control Orders or compliance notifications; and

- Any other relevant reports, such as reports of previous optimization studies.

A site inspection of the works should be undertaken as early as possible in the project, possibly in conjunction with the kick-off meeting. This site inspection should be led by knowledgeable members of the operations staff. One objective of this site inspection is to communicate operations staff's knowledge of specific issues related to the works such as hydraulic bottlenecks, mechanical equipment deficiencies, plant modifications not shown on as-built or record drawings, or other operational concerns that may be relevant to the investigation. This hands-on information can provide insights into factors that limit plant performance or capacity that may not be evident from the historic data. At the same time, the optimization team can collect information regarding sampling locations, can visually inspect flow meters for design or installation problems that may affect the validity of flow data, can determine the types and locations of on-line instruments and data acquisition equipment that might be used during subsequent field investigations, and can make other relevant observations to support the historic data review.

The historic data review provides a summary of the historic operating conditions, unit process loadings and unit process performance, including but not limited to:

- Graphical presentations of historic flows, raw sewage and final effluent quality, including a comparison to the design basis or C of A requirements;

- Analysis of effects of seasonal or other factors on sewage flows and loadings;

- Review of unit process loadings and performance, and comparison to typical and/or design loadings and performance; and

- Identification of performance or operational issues evident from the site inspection or from the historic data.

The historic data review should present a description of the works, with a process flow diagram (PFD) and a summary of the key design criteria for major unit processes. Examples of a typical PFD and a tabular summary of process design criteria are shown in Figure 5-2 and Table 5-1, respectively.

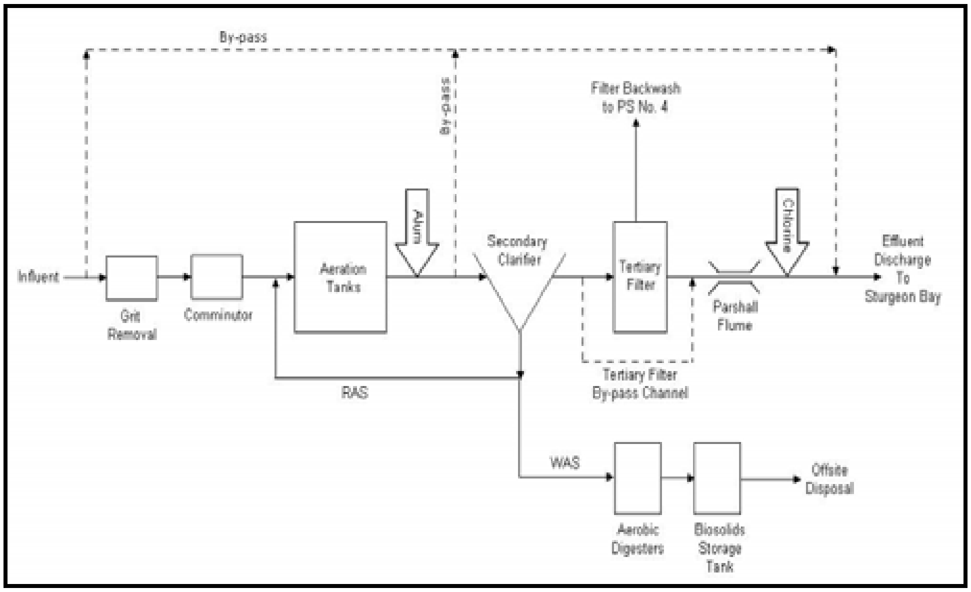

Figure 5-2 - Example Process Flow Diagram (PFD)

Table 5-1 - Example of Process Design Summary

Unit Process	Design Parameter
Inlet Works	
Grit Removal	
Type	Grit Channels
Number	2
Length x Width	6.5 m x 0.75 m
Flow velocity	0.3 m/s
Hydraulic Capacity	13,400 m^3/d
Comminutor	
Number	1 (w/ bypass channel)
Aeration Tanks	
Number	3
SWD	3.7 m
Length x Width	30 m x 5 m
Volume, each	558 m^3
Volume, total	1675 m^3
Type of Aeration System	Fine Bubble Diffusers
Blowers	
Number	3
Type	Rotary Positive Displacement
Capacity, each	1,105 m^3/hr @ 48kPa
Firm Blower Capacity (aeration and digestion)	2,210 m^3/hr @ 48kPa

Table 5-1 - Example of Process Design Summary (continued)

Unit Process	Design Parameter
Secondary Clarifiers Type Number SWD Diameter Surface Area	 Circular 1 4 m 16 m 200 m^2
Effluent Filter Number Type Length x Width Surface Area Peak Design Rate Peak Capacity	 1 Travelling Bridge 12.2 m x 3.8 m 46 m^2 6 m/h 6,600 m^3/d
RAS / WAS Pumping Number Type Capacity, each Firm Pumping Capacity (RAS) Firm Pumping Capacity (WAS)	 3 centrifugal 2 pumps rated at 2,366 m^3/d @ 5 m TDH 1 pump rated at 1,964 m^3/d @ 5.8 m TDH 2,366 m^3/d @ 5 m TDH 1,964 m^3/d @ 5 m TDH
Effluent Flow Meter Type	V-notch weir
Outfall Land Portion Length Land Portion Diameter Marine Portion Length Marine Portion Diameter Hydraulic Capacity (with 8 diffuser ports open) Number of Ports Currently Open	 130 m 750 mm 600 m 560 mm 12,184 m^3/d 5 of 8
Alum Addition (for Phosphorus Removal) Number of Pumps Pump Capacity, each Chemical Storage Capacity	 2 390 L/d 23 m^3

Table 5-1 - Example of Process Design Summary (continued)

Unit Process	Design Parameter
Disinfection (Chlorination)	
Number of Pumps	2
Pump Capacity, each	780 L/d
Chemical Storage Capacity	11 m^3
Chlorine Contact Time in Outfall	40 min @ 6,600 m^3/d
Aerobic Digestion	
Number	2
SWD	3.5 m
Length x Width	11.7 m x 5 m
Volume, each	205 m^3
Volume, total	410 m^3
Type of Aeration System	Coarse Bubble
Biosolids Storage Tank	
Number	1
SWD	3.5 m
Length x Width	6 m x 5 m
Volume	105 m^3
Type of Aeration System	Coarse Bubble
Notes: RAS – return activated sludge; SWD – side water depth; TDH – total dynamic head; WAS – waste activated sludge	

A separate table should summarize key design parameters such as the design average daily flow (ADF), peak flow, design wastewater strengths or loadings, and the effluent quality requirements (objectives and compliance limits) specified in the C of A.

A key element of the historic data review, when the overall performance of the STP is being assessed, is a mass balance on solids in the works, often called a Sludge Accountability Analysis. Sludge production data recorded in the historic plant operating files for raw primary sludge, waste biological sludge, and processed sludge hauled off-site for land application or disposal should be compared with typical sludge production data for a similar type of plant from sources such as MOE (2008), WEF/ASCE (1998), and Metcalf & Eddy (2003). Lack of agreement to within a reasonable error band (+/- 15 to 20 percent) may suggest that flow metering and/or sampling data are not reliable or that solids recycle from sludge processing units is affecting the data. This information can be used to develop a field monitoring program that would focus on identifying the cause of the sludge mass balance anomaly.

5.2.3 Process Capacity Assessment

A process capacity chart is a common means of graphically illustrating the capacity of individual unit processes comprising the works. The key operating parameters of a unit process are compared to typical design parameters from sources such as MOE (2008), WEF/ASCE (1998), and Metcalf & Eddy (2003). Based on this comparison, a bar chart is prepared showing the estimated capacity of each unit process in comparison to the design capacity of the works. Those unit processes with the lowest capacity represent the capacity limiting unit processes and should be candidates for optimization if increased capacity is an objective of the study. These capacity limiting processes may also be the processes that limit the performance of the works. An example of a process capacity chart is presented in Figure 5-3.

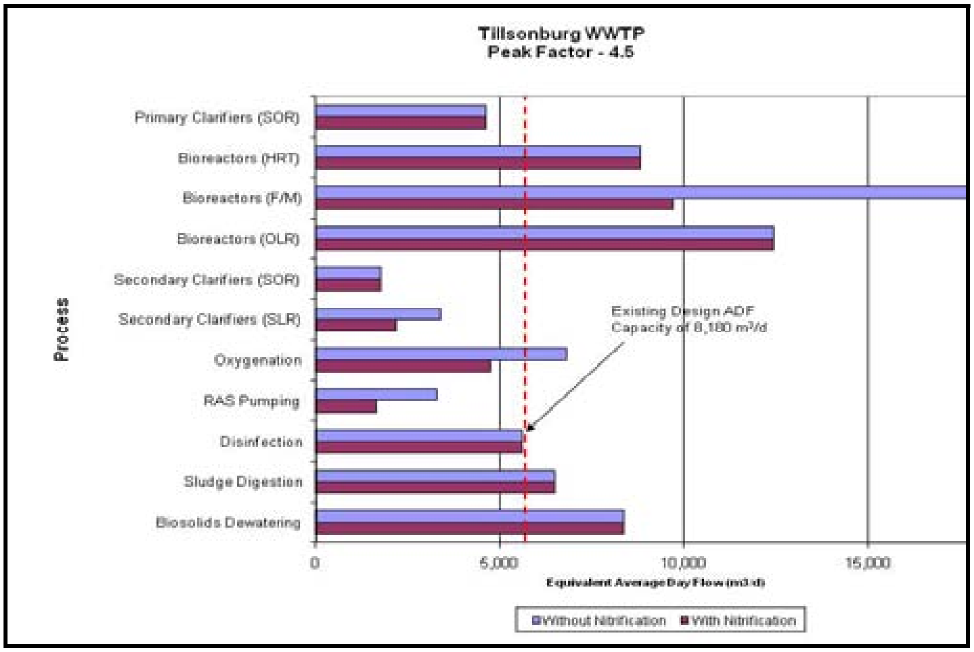

Figure 5-3 - Example Process Capacity Chart

The CPE component of the CCP (Chapter 4) uses a similar approach to graphically illustrate the performance limiting unit processes. The Ontario guidance manual for conducting CCPs (Wastewater Technology Centre and Process Applications Inc., 1996) contains comparative capacity values that can be used to assess the capacity of select unit processes based on Ontario experience.

The capacities illustrated in the process capacity chart are based on comparisons to generally accepted design guidelines or criteria. These theoretical capacities can be used to prioritize the specific field and/or modelling investigations that will be undertaken during subsequent phases of the optimization project. For example, if the desk-top analysis suggests that secondary clarifiers have lower capacity than other unit processes comprising the liquid treatment train, clarifier stress tests could be undertaken to confirm the capacity of the clarifiers. Process modelling and simulation (Chapter 6) can also be used to refine a capacity estimate that is based on typical design values. The results of the field and/or modelling investigations can then be used to develop a case-specific estimate of the actual capacity of the various unit processes. The field investigations should be designed to ensure that any possible impact of optimizing a specific unit process on the operation or capacity of other processes is considered.

5.2.4 Field Investigations

At the completion of Phase 1 of the investigation, a detailed Phase 2 Work Plan for field investigations and other studies aimed at confirming the Phase 1 findings can be developed, along with a schedule and cost to undertake these activities. The scope of the investigations, along with the schedule and cost, should be communicated to all participants in the study at a Phase 2 kick-off meeting. Scheduling the field work is an important agenda item at this meeting as other plant activities and the possible impact on the field testing should be carefully considered (Section 5.3.5). Other items outlined in Section 5.3 should also be discussed at the Phase 2 kick-off meeting.

Depending on the findings of Phase 1, field investigations may include, but are not necessarily limited to, any or all of the following activities. The details of these field investigations are discussed elsewhere in the Guidance Manual.

- Intensive On-Line or Off-Line Monitoring.
- Operator Training.
- Flow Meter Evaluation.
- Hydraulic or Process Modelling.
- Stress Testing.
- Oxygen Transfer Testing.
- Mixing or Tracer Tests.
- Pilot-scale or Full-scale Testing.

The objectives of the field investigations are:

- To verify the findings of the Phase 1 investigations with actual field test data;
- To identify cost effective approaches to mitigate performance or capacity limitations in the works; and
- To document potential benefits.

5.2.5 Stress Testing

Stress testing can be an important component of field investigations and is aimed at establishing a plant-specific estimate of the capacity of a unit process by increasing the loading to the process and measuring the process response.

Typically, unit processes are designed based on accepted design criteria (MOE, 2008; WEF/ASCE, 1998; Metcalf & Eddy, 2003). These design criteria may under-estimate the capacity of a unit process. Under normal conditions, a unit process is seldom operated intentionally at loadings beyond the design level. During stress testing, loadings are artificially increased for a pre-determined time period to assess the capability of the unit process to operate at loadings higher than design without deterioration in performance. Alternatives that can be used to increase the loadings to a specific unit process include:

- Reducing the number of process units (e.g. clarifiers, screens, UV lamps) in service, thereby increasing the loading on the units in service;
- Diverting part of the flow to the process unit being tested in situations where there are multiple units operating in parallel (e.g. filters, screens, clarifiers);
- Recirculating treated effluent to a unit process to increase the hydraulic loading, if hydraulic loading is a key design parameter;
- Testing under wet weather flow conditions when sustained periods of high flow are often experienced; and/or
- Testing during peak diurnal loading periods when flows and loads are typically highest.

In planning stress tests, all key design parameters that affect the performance of a unit process must be increased to assess the impact. Table 5-2 presents the critical loadings that should be considered in stressing various unit process types.

The duration of the stress test should be adequate to ensure that a response will be observed. For example, stress testing of unit processes such as clarifiers or filters may require a few days as the response to hydraulic loading increases is relatively rapid. Stress testing of biological processes will require operation for an extended period of time (up to a year) because of the effect of seasonality on biological process performance and the long (two to three SRTs) response time of these unit processes.

A detailed monitoring program should be developed to ensure that the response of the unit process being tested can be quantified. This may require intensive sampling of the process effluent over a short period of time (i.e. hourly samples). It is also important that the flows and loads to the unit process under investigation be monitored and recorded.

Stress testing may result in deterioration in effluent quality as loads are increased to the unit process under test. Contingency plans should be developed that specify what steps will be taken in the event of effluent quality deterioration (Section 5.3.1). MOE should be notified and made aware of the specific testing being conducted.

Clear objectives should be established prior to stress testing that identifies how the capacity of a unit process will be determined. To ensure a margin of safety in establishing process capacity, it is recommended that C of A objectives rather than compliance limits be used as the performance requirement where applicable. For example, during stress testing of secondary clarifiers or filters, the target performance should be the effluent total suspended solids (TSS) or total phosphorus (TP) objective set in the C of A or the expected objectives in a new C of A if the plant is being re-rated.

More detail on stress testing of individual unit processes is presented in subsequent chapters of the Guidance Manual.

Table 5-2 - Stress Test Parameters for Selected Unit Processes

Unit Process	Design Parameter
Screening, Grit Removal	• Hydraulic loading
Primary Clarifier	• Surface overflow rate (SOR) • Hydraulic retention time (HRT)
Secondary Clarifier	• SOR • Solids loading rate (SLR)
Bioreactor	• Solids retention time (SRT) • Food-to-microorganism ratio (F/M_v) • Organic loading rate
Effluent Filter	• Hydraulic loading • Solids loading rate
Chlorination	• HRT • Chlorine residual

5.2.6 Modelling and Simulation

Process or hydraulic models of the treatment process, or individual treatment units, can be developed and calibrated based on historic data and the results of field investigations, if applicable. These models can then be used to project the impact of increased flows or loadings on the existing treatment systems, or to project the impact of physical upgrades or process changes on performance (such as installing baffles in a clarifier, or modifying cycles times in a sequencing batch reactor (SBR) system). More detailed information regarding the application of modelling and simulation as part of the optimization process is presented in Chapter 6.

5.2.7 Reporting

As a minimum, reports or Technical Memoranda (TMs) should be prepared at the completion of Phase 1 and Phase 2 of the optimization project. The Phase 1 TM should summarize the findings of the historic data review and the process capacity assessment, and present a recommended Phase 2 Work Plan with estimated budget and cost, including the support required from operations staff and the estimated time commitment.

The Phase 2 TM should summarize the findings of the field investigations, present the implementation plan for any process changes or physical upgrades recommended to mitigate capacity or performance limitations in the works, and document the potential benefits. Depending on the scope of the study and its duration, individual TMs could also be prepared summarizing the findings of specific investigations. For example, a TM could be prepared to present the findings of oxygen transfer testing or stress testing.

Workshops with the owner and operations staff can be an effective tool to disseminate the findings at key points in the project. For example, a Workshop at the completion of Phase 1 provides an opportunity for the participants to comment on the findings of the desk-top review based on hands-on knowledge of the works and to provide input to the scope of the field investigations in Phase 2. A Workshop at the completion of Phase 2 provides the participants with an opportunity to comment on the proposed upgrades to the works and the implementation plan.

A detailed discussion of the reporting component of a sewage works optimization program is presented in Chapter 21.

5.3 OTHER CONSIDERATIONS

5.3.1 Notifications

Some field testing, such as stress testing has the potential to adversely affect effluent quality. Tracer tests can result in a discolouration of the plant effluent. Some demonstration or pilot testing may require approval by the MOE and a temporary C of A if the works are operated in a manner different than specified in the sewage works C of A. It is important that the MOE be aware of the specifics of field testing programs and that any necessary approvals for testing are obtained prior to testing. The owner should consult with MOE prior to initiating optimization studies to determine what, if any, approvals are needed.

For stress tests, it is important that a Contingency Plan is prepared that clearly identifies the steps that will be taken in the event that there is a deterioration in effluent quality as a result of the test. Clearly identified avenues of communication and decision making should be identified in the Contingency Plan.

5.3.2 Responsibilities of Parties

The Work Plan for the Phase 2 field investigations should clearly identify who will be responsible for operation of the process during the test, sample collection and monitoring, submittal of samples with related Chain of Custody forms to the analytical laboratory, and reporting of results. The Work Plan should also clearly state the conditions under which the test will be conducted, the sampling frequency, the analytical program, and the test schedule and duration. Operation of treatment units during all field testing programs must be the responsibility of licensed operators as stipulated under O. Reg. 129/04.

5.3.3 Health and Safety Requirements

Some field tests involve hazardous chemicals or activities with potential safety hazards. All staff involved in the field test program should be adequately trained, have access to the necessary Workplace Hazardous Materials Information System (WHMIS) information and material safety data sheet (MSDS) for any chemicals being used, and be familiar with the safety hazards. A site-specific Health and Safety Plan should be developed and circulated to all parties involved in the program prior to the testing.

5.3.4 Analytical Quality Assurance (QA) Considerations

Many of the field tests require analytical support. An analytical laboratory accredited by Standards Council of Canada (SCC) should be used for any compliance testing and should be considered if the data are intended to support an application to re-rate the sewage works. The sewage treatment plant process laboratory can perform routine tests to obtain rapid turn-around for process control purposes.

A Quality Assurance (QA) program should be incorporated into the analytical program, including in the range of 5 to 10 percent blanks, duplicates and spiked samples.

5.3.5 Scheduling Considerations

To the extent possible, field testing should be scheduled to avoid process or equipment maintenance shut-downs that might affect the test. Holidays and vacation periods should be avoided for short-term, high intensity tests such as clarifier dye tests or stress tests. Prior to initiating the field program, the schedule of key maintenance activities for the duration of the test should be obtained from and discussed with plant operations and maintenance staff.

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CHAPTER 6

MODELLING AND SIMULATION

CHAPTER 6

MODELLING AND SIMULATION

6.1 OVERVIEW OF MODELLING AND SIMULATION

A model is a set of mathematical relationships that are used to describe physical, chemical and biochemical interactions. In some cases, the mathematical relationships that form a model can be quite simplistic, as is the case when describing the concentrations of substrate in a complete mix reactor. In other cases, the model can be quite complex and involve multiple interacting relationships, such as models that describe the biochemical reactions in a biological nutrient removal (BNR) process. The level of model complexity required often depends on the modelling objectives.

Models require calibration and validation to ensure that they provide meaningful results. Calibration involves modifying model parameters so that the model output matches actual field measurements. Validation involves running a series of model calculations using field data independent from those used for calibration, and comparing the model output to actual field results. If the output during validation matches the actual field results, the model can be assumed to be properly calibrated.

Simulators are computer programs which use a model, or set of models, as a basis for calculations. The user can configure the simulator to describe the physical layout of a treatment plant or specific unit processes within the plant. The simulator can be used to perform simulation runs at various operating conditions to identify impacts on process performance. Some simulators allow both steady-state (static) and dynamic (time varying) simulations.

Specific applications of modelling and simulation relevant to wastewater treatment are presented in Section 6.2.

6.2 APPLICATIONS OF MODELLING AND SIMULATION

6.2.1 Biological Process Modelling

Overview and Use

Biological models are a set of mathematical equations that describe the biological interactions between various types of microorganisms involved in secondary wastewater treatment. Most biological models in use today are based on the International Water Association (IWA) activated sludge models: ASM1, ASM2, ASM2d, and ASM3. More information regarding these models can be found in IWA (2000).

Biological process models can be used to:

- Determine a facility's biological treatment capacity;
- Identify components of the biological system that are capacity limiting;
- Predict the impact of operational changes on system performance; and
- Determine optimal upgrade requirements to increase capacity and/or treatment efficiency while minimizing costs.

There are various simulator packages available commercially, some of which are identified in Table 6-1. Many of these simulators also incorporate simplified one-dimensional clarifier models (Section 6.2.3), in addition to chemical phosphorus removal, attached growth systems, and digestion models, so that "whole plant" simulations can be run to predict clarifier effluent quality at various operating conditions. Some of these simulators can also model the chemical reactions occurring in the bioreactor during simultaneous chemical phosphorus removal (Chapter 16), allowing effluent total phosphorus (TP) concentrations to be predicted.

Table 6-1 - Commercial Biological Process Model Simulator Packages

(Adapted from WERF, 2009)

Simulator	Vendor	Main Users	Primary Area of Use
BioWin ™	EnviroSim	Consultants	Worldwide
GPS-X ™	Hydromantis	Consultants	Worldwide
STOAT ™	WRc	Consultants / Owners	US, UK
WEST ™	Hemmis	Academia / Consultants	Worldwide
SIMBA ™	Ifak System	Academia	Germany, Holland
ASIM ™	Holinger	Academia	Worldwide

Biological process simulators can be used to estimate the biological treatment capacity of an existing treatment process. In addition, biological process simulators can be used to predict the impact of operational parameters, such as solids retention time (SRT), hydraulic residence time (HRT), dissolved oxygen (DO), temperature or mixing configuration, or upgrades on system performance and capacity.

It is important to recognize that the models used as a basis for these calculations may have less inherent safety margin than typical design guidelines (Section 6.3.1). If steady-state simulations are used, the model may over-predict actual treatment capacity (Merlo *et al*., 2009). Dynamic modelling may give a more representative prediction of system performance: however, dynamic modelling may not always be feasible, due to the data available for calibration, limitations of the model and/or the type of system being modelled. Therefore, the results of biological system modelling should be confirmed through on-site testing. If biological modelling is being used as a basis for the design of process upgrades or STP re-rating, a safety factor should be applied to ensure the required effluent quality can be consistently met.

Calibration and Validation

Generally, historic plant operating data, including raw wastewater and final effluent characteristics and bioreactor operating conditions (SRT, mixed liquor suspended solids (MLSS), mixed liquor volatile suspended solids (MLVSS), etc.), are used for calibration and validation. A statistical analysis of the operating data should be performed to identify potential outliers or other inconsistencies. An assessment of data gaps should also be completed to identify any additional sampling or monitoring that may be required to properly calibrate and validate the model.

Nitrifier growth rates have been found to vary significantly from one STP to the next (WERF, 2003). If it is suspected that the nitrifier growth rate parameter needs to be modified from its default value, nitrifier growth rate testing can be performed to determine the site-specific value (WERF, 2003).

For most municipal wastewater treatment applications, default values for raw wastewater stoichiometric and other microorganism kinetic parameters should be adequate. These parameters may need to be adjusted for municipal wastewater treatment facilities that have a large industrial input.

There are various protocols available for the calibration and validation of biological process models, including:

- Hochschulgruppe (HSG) guidelines (Langergraber *et al*., 2004);
- STOWA protocol (Hulsbeek *et al*., 2002);
- BIOMATH protocol (Vanrolleghem *et al*., 2003); and
- WERF Methods for Wastewater Characterization in Activated Sludge Modelling (WERF, 2003).

A comparison of these protocols is provided in WERF (2009).

6.2.2 Hydraulic Modelling

Overview

Hydraulic models describe the characteristics of flow over and through control devices, such as weirs, gates and flumes. Hydraulic models are used to develop Hydraulic Grade Lines (HGLs) and Energy Grade Lines (EGLs), which describe flow through a STP. Model development is based on fluid mechanics theory and hydraulic equations.

Hydraulic modelling can be used to:

- Determine a facility's hydraulic capacity;
- Identify hydraulic bottlenecks;
- Identify locations of flow imbalances; and
- Identify optimal locations for chemical addition to promote mixing and flocculation.

The first step in developing a hydraulic model is to identify the hydraulic elements within the facility. Table 6-2 lists common hydraulic elements in STPs. On-site measurements and surveying may be required to confirm the dimensions and elevations of hydraulic elements, channels, piping and other structures shown on plant record drawings.

Table 6-2 - Common Hydraulic Elements in Sewage Treatment Plants

(Adapted from MOEE *et al.*, 1996)

Type	Element	Modelling Technique
Pressurized Flow	• Pipe	• Hazen Williams equation • Darcy Weisbach equation • Minor-loss coefficients
Open Channel	• Rectangular • Trapezoidal • Circular	• Manning's equation • Gradually varied flow
Distribution / Collection	• Collection Channel • Distribution Channels • Launders • Diffusers	• Modified Manning's equation • Gradually varied flow • Orifice equations • Minor-loss coefficients

Table 6-2 - Common Hydraulic Elements in Sewage Treatment Plants (continued)

(Adapted from MOEE *et al*., 1996)

Type	Element	Modelling Technique
Transitional	• Abrupt or Gradual Change in Cross-section • Bends	• Gradually varied flow • Minor-loss coefficients
Obstruction	• Bar Screen • Trash Rack • Control Valves	• Manufacturer's information • Ratio of area
Gates	• Sluice Gates • Submerged Orifices	• Sluice gate equations • Orifice equations
Flumes	• Parshall, Leopold -Lagco, etc.	• Flume equations, specific to the flume configuration and dimensions
Weirs	• Rectangular, V-Notch, etc. • Side Flow – Bypass Channels	• Weir equations, specific to the weir configuration and dimensions • Spatially varied flow

Under normal conditions, influent flows to a STP change very gradually. As a result, steady-state hydraulic calculations can be used, greatly simplifying the complexity of the required calculations.

To develop a STP's hydraulic profile, calculations begin at the most downstream flow control element, and move progressively upstream. Hydraulic profiles should be developed at various influent flow rates. More information regarding the development of hydraulic profiles can be found in WERF (2009), Nicklow & Boulos (2005), and MOEE *et al.*, (1996).

There are very few commercially available hydraulic modelling software packages. In most cases, a spreadsheet program or computer programming languages are used to develop hydraulic models on a case-by-case basis.

Calibration and Validation

Model calibration and validation is based on liquid level data collected at a minimum of two different flow rates. It is recommended that liquid levels be measured at both the average dry weather flow rate and peak flow rate. Liquid levels should be measured at various locations throughout the plant. In addition, all measured flow data, including recycle flow rates, should be recorded. The measurement of flow splits between various treatment trains can also be recorded by using temporary flow meters or dye dilution techniques (Section 9.1).

The model can then be calibrated, based on recorded liquid levels at a given flow rate, by modifying assumptions and hydraulic element parameters and coefficients so that the model output matches the field data. The second set of liquid levels can then be used to validate the model.

Details regarding the calibration and validation of hydraulic profiles can be found in MOEE *et al*. (1996) and Nicklow and Boulos (2005).

6.2.3 Clarifier Modelling

Overview

Clarifier hydrodynamic models describe the characteristics of flow and solids settling that take place within a clarifier. Development of clarifier models is based on fluid dynamics, solids flux theory, and the physical configuration of the subject clarifier(s).

Clarifier hydrodynamic modelling can be used to:

- Determine a clarifier's hydraulic capacity;
- Predict the impact of operational changes on clarifier performance; and
- Determine optimal baffling, inlet structure, and weir configurations to improve clarifier performance.

Clarifier hydrodynamic models can be divided into three types, namely one-dimensional (1-D), two-dimensional (2-D), and three-dimensional (3-D).

Generally, 1-D models are based on flux theory (Section 14.4.2). Only the settling processes that occur in the vertical direction are modelled, as it is assumed that the horizontal velocity and concentration profiles are uniform. These models can be calibrated with actual plant data, and can provide a good representation of solids inventory within the system. However, the 1-D model cannot take into account influences from tank geometry, sludge removal processes, density currents or short - circuiting. Due to its simplistic nature, a 1-D model may only be capable of identifying a settling problem within a clarifier; more detailed 2-D or 3-D modelling may be required to identify the nature and cause(s) of the problem.

2-D models take into account flux theory, entrance and exit effects, and sludge removal processes. Only the settling and flow processes that occur in the vertical and horizontal (from clarifier entrance to exit) directions are modelled, as it is assumed that the flow characteristics within the clarifier are consistent across all cross sections perpendicular to the bulk flow. 2-D models are reported to give reasonably good predictions of behaviour of circular clarifiers and some rectangular clarifiers, and can therefore be used to estimate the impact of baffle installation or modification on clarifier performance. A 3-D model may be required for circular clarifiers that are subject to asymmetric flow due to high winds or the configuration of the inlet port or effluent weirs, or rectangular clarifiers with non-uniform lateral feed. Square clarifiers often exhibit strong 3-D flow behavior and, as such, a 2-D model may not be capable of providing sufficient information regarding the flow characteristics within these clarifiers.

3-D models take into account flux theory, entrance and exit effects, sludge removal processes, and variations in flow patterns in all three dimensions. Although these models provide detailed information regarding the characteristics of flow within the clarifier, they require a great deal of computing power.

Table 6-3 presents a summary of the applicability of the various types of clarifier models. In general, a simulator computer program utilizes computational fluid dynamics (CFD) theory to solve the model's system of equations. More information regarding the theory and mathematics of the 1-D, 2-D, and 3-D models can be found in Ekama *et al.* (1997).

Table 6-3 – Selection of Appropriate Clarifier Model
(Adapted from Ekama *et al.*, 1997)

Modelling Objective / Application	Minimum Model Required
System mass inventory assessment.	• 1-D clarifier model coupled with a biological process model; and/or • 2-D clarifier model coupled with a biological process model
Sludge blanket level assessment.	• 1-D clarifier model coupled with a biological process model; and/or • 2-D clarifier model coupled with a biological process model
Sludge withdrawal scheme assessment.	• 2-D clarifier model coupled with a biological process model; and • Possible confirmation of 2-D modelling results with a 3-D clarifier model
Optimization of tank geometry.	• 2-D clarifier model; and/or • 3-D clarifier model
Retrofitting clarifier(s) with appurtenances, including baffles.	• 2-D clarifier model; and/or • 3-D clarifier model
Tanks subject to strong three dimensional flow behaviour due to, for example, wind shear, inlet / outlet configuration, and/or non-uniform lateral feed in rectangular clarifiers.	• 3-D clarifier model
Assessment of density currents.	• At least a 2-D clarifier model

Calibration and Validation

1-D models are generally calibrated and validated based on plant operational data and solids flux information. Information regarding the development of solids flux curves is presented in Section 14.4.2.

In addition to the information required for 1-D model calibration and validation, 2-D models also require information regarding velocity and solids profiles within the clarifier at various flow rates. These profiles can be obtained through dye testing (Section 14.4.1).

The information required for the calibration of 3-D models is similar to that required for 2-D models; however the three-dimensional aspects of the intra-clarifier velocity and solids profiles need to be taken into consideration during on-site testing. These data can be gathered through dye testing (Section 14.4.1). In this way, calibration and validation can be done based on three-dimensional data.

6.2.4 Modelling Reactor Flow Characteristics

The two simplest models that can be used to describe flow through a reactor are the "complete mix model" and the "plug flow model". In a complete mix reactor, it is assumed that the composition of the reactor contents is homogenous throughout the reactor volume, and that mixing of the influent is done instantaneously. In a plug flow reactor, it is assumed that all influent to the reactor has the same residence time, and that the flow moves as a "plug" down the length of the reactor. Therefore, in a plug flow reactor, the composition of the reactor contents varies in the direction of flow.

In practice, full scale reactors only approximate the behaviour of complete mix or plug flow reactors due to flow non-idealities, such as dead zones, short-circuiting, and longitudinal dispersion in plug flow reactors. Tracer testing can be used to identify and quantify the effects of these flow non-idealities. Information regarding tracer testing methods and data analysis is presented in Metcalf & Eddy (2003). Depending on the objectives of the reactor modelling and/or the severity of flow non-idealities, complete mix and plug flow models can provide a good approximation of reactor flow characteristics.

Typical examples of reactors in STPs that approximate complete mix characteristics can include primary digesters and complete mix bioreactors. Examples of reactors that approximate plug flow characteristics can include ultraviolet (UV) reactors, chlorine contact tanks, and plug-flow bioreactors.

If more detailed information is required for complete mix reactors, mixing modelling can be used to describe the reactor hydrodynamics. This is explained in more detail in Section 6.2.5.

The behaviour of plug flow reactors can be approximated by modelling several complete mix reactors operating in series. The number of complete mix reactors to be used in the model depends on the geometry of the plug flow reactor, the flow rate through the reactor, and any known flow non-idealities. Such approximations can be used in combination with a biological process model to simulate the retention time distribution provided within a plug flow bioreactor.

A 2-D or 3-D model would be required to identify the causes of flow non-idealities, and to evaluate alternative options to optimize the plug flow behaviour of the reactor, such as baffle installation or modification of inlet and/or outlet structures. In general, a simulator computer program utilizes CFD theory to solve the model's system of equations, and to allow for the user to modify reactor configuration to model the impact on mixing performance.

6.2.5 Mixing Modelling

Overview

Hydrodynamic mixing models describe the characteristics of flow and suspended solids mixing that take place within a mixed reactor, such as a digester or a complete mix aeration tank. The development of mixing models is based on fluid dynamics, including rheology of the reactor contents, and the physical configuration of the subject reactor.

Hydrodynamic mixing modelling can be used to:

- Identify potential dead-zones within a mixed reactor;
- Identify potential short-circuiting within a mixed reactor;
- Predict the impact of operational changes on mixing performance; and
- Determine optimal baffling and mixer configurations to improve performance.

Mixing modelling is generally accomplished through the use of 3-D models. In general, a simulator computer program utilizes CFD theory to solve the model's system of equations, and to allow for the user to modify reactor configuration to model the impact on mixing performance.

The presence of dead-zones and/or short-circuiting within a complete mix reactor reduces the effective reactor volume available, thus reducing the effective treatment capacity. In such cases, the mixing efficiency can be optimized by making adjustments, such as installation of baffles, addition or modification of mechanical mixers, and/or inlet and outlet structure modifications. Mixing modelling can be used to evaluate the impact of these changes on process performance and to select the optimal upgrade approach.

Calibration and Validation

In addition to the geometry of the reactor, the results of tracer testing can be used to calibrate and validate a mixing model. During tracer testing, samples would need to be collected at various locations within the reactor, at various time intervals, to provide sufficient data points for proper calibration and validation.

6.3 LIMITATIONS OF MODELLING AND SIMULATION

6.3.1 Safety Factors

Due to the nature of the mathematical relationships used, models may have less inherent safety margin than typical design guidelines. Dynamic modelling may give a more representative prediction of system performance than steady-state modelling; however, dynamic modelling may not always be feasible, due to the data available for calibration, limitations of the model and/or the type of system being modelled. As a result, a separate safety factor should be applied to designs based on modelling results and/or field testing should be completed to confirm the modelling results.

6.3.2 Quality of Data

The accuracy of a model depends on the quality of the data used in its development, calibration, and validation. For this reason, all data should be screened to identify any outliers or other inconsistencies, and to identify any data gaps that would require additional data collection.

6.3.3 Improper Calibration

Improper calibration occurs when key model parameters are incorrectly adjusted to match actual field data.

During model calibration, it is possible to adjust model parameters to make the simulator output match actual field data, however this alone does not ensure that the model is accurately describing the actual behaviour within the system. If improperly calibrated, the model would not be able to predict system behaviour for any conditions other than those used for calibration.

6.4 CASE HISTORIES

6.4.1 Utilization of Modelling to Optimize Clarifier Performance During Wet Weather Conditions

The following case study is based on information presented in Griborio *et al*. (2008).

Background and Objectives

A 56,800 m^3/d (15 mgd) secondary STP was subject to high flows during wet weather periods, with peak flows as high as 142,000 m^3/d (37.5 mgd). The STP was equipped with two 39.6 m (130 ft) diameter circular secondary clarifiers, each with a 3.7 m (12 ft) side water depth (SWD), and a feed/flocculation well 5.8 m (19 ft) in diameter and 1.2 m (4 ft) deep. In addition, construction of a third circular secondary clarifier, with a diameter of 48.8 m (160 ft) and a SWD of 4.6 m (15 ft), was proposed.

Clarifier modelling was used to develop recommended clarifier modifications and operational strategies to enhance secondary effluent quality, in terms of total suspended solids (TSS) concentration, during wet weather events.

The secondary treatment system was modelled using a combination of: a quasi-three-dimensional clarifier model developed at the University of New Orleans (commonly referred to as the 2Dc model) and BioWin™, a biological process modelling and simulation program to model the secondary treatment system.

Optimizing Existing Clarifier Performance During Wet Weather Flows

Three optimization strategies were evaluated, alone and in combination, namely:

- Operating the bioreactors in step-feed mode to reduce the solids loading to the clarifiers;
- Retrofitting the existing clarifiers with enlarged centre feed / flocculation wells; and
- Adding polymer to enhance the settling properties of the mixed liquor.

Table 6-4 presents the model-predicted secondary clarifier effluent TSS concentrations with and without the optimization strategies.

Table 6-4 – Model-Predicted Secondary Effluent Quality

Optimization Strategy(ies) Modelled	Effluent TSS Concentration (mg/L)
None.	162
Step-feed operation of the bioreactors.	55
Step-feed operation of the bioreactors; and Enlarging the inlet/flocculation well.	12.5
Step-feed operation of the bioreactors; and Polymer addition.	25
Step-feed operation of the bioreactors; Enlarging the inlet/flocculation well; and Polymer addition.	10

Based on the modelling results, implementation of step-feed operation of the bioreactors significantly reduced effluent TSS concentrations. Enlarging the inlet/flocculation well also significantly reduced the effluent TSS concentrations, while the addition of polymer had a modest impact on performance.

6.4.2 Use of Modelling for MBR Process Design Optimization

The following case study is based on information presented in Latimer *et al*., (2008).

Background and Objectives

The Big Creek Water Reclamation Facility (WRF) is an extended aeration facility with tertiary deep bed filters and UV disinfection. The facility is undergoing an expansion from 91,000 m^3/d (24 mgd) to 144,000 m^3/d (38 mgd); however, the existing site has limited room available for expansion.

As part of the expansion, the existing treatment train will be retained, and a new 53,000 m^3/d (14 mgd) membrane bioreactor (MBR) treatment system will be constructed. The MBR will be designed to provide both biological nitrogen and biological phosphorus removal. The two treatment trains would operate in parallel. In addition, new primary clarifiers would be constructed, and would be common to both treatment trains.

BioWin™ biological process modelling software was used to evaluate various expansion alternatives in order to determine the optimal design for the MBR treatment train. Provision was to be made in the design to allow for future conversion into a full MBR plant that would be capable of meeting a future total nitrogen (TN) limit of 5 mg/L.

Model Development and Calibration

The results of intensive sampling, along with existing plant data, were used to calibrate a BioWin™ model of the existing treatment system. Results of clarifier stress testing and sludge settleability information were also used to develop and calibrate a CFD model of the existing clarifiers. The BioWin™ and clarifer models were used together to evaluate expected process performance for various design alternatives.

MBR Process Optimization

The membrane tanks in submerged membrane-type MBR systems operate at high dissolved oxygen (DO) concentrations (> 4 mg/L) due to the requirement for air scouring. Due to the high rate of return of mixed liquor required from these membrane tanks (on the order of 4 to 5 times the influent flow), the DO recycled back from these membrane tanks can negatively impact biological nitrogen and phosphorus removal systems by introducing oxygen into zones which are to be operated in anaerobic or anoxic mode.

Modelling was conducted to mitigate the impact of the membrane tank return stream on biological phosphorus removal, thereby reducing the required bioreactor volume and providing a more stable treatment process. Simulations were run for various sludge return and internal recycle configurations to determine the optimal configuration. The model-predicted optimal configuration is presented in Figure 6-1.

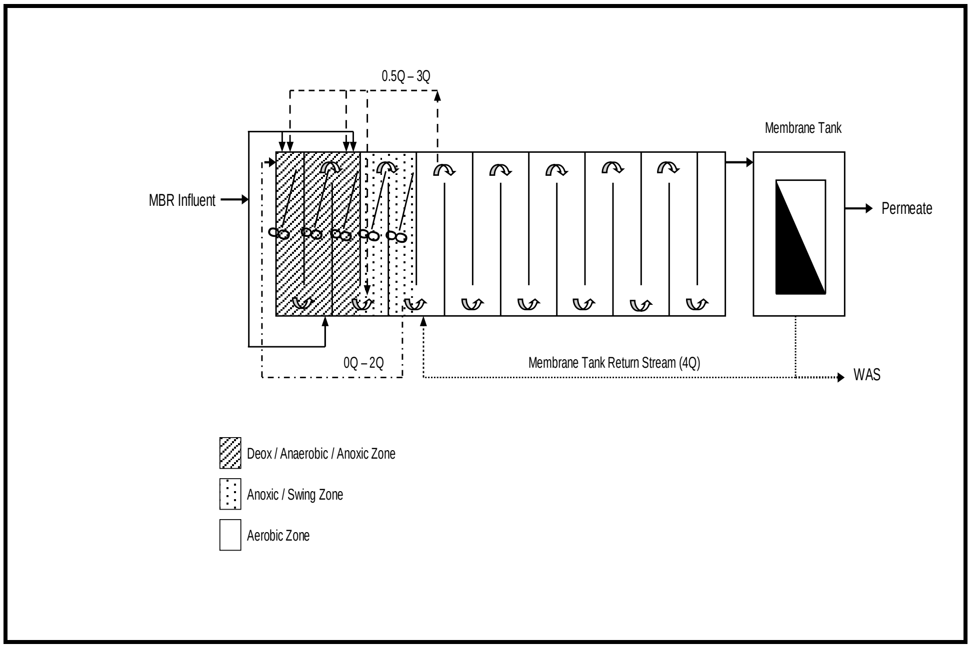

Figure 6-1 – Optimal MBR Internal Return Stream Process Configuration

(Adapted from Latimer *et al.*, 2008)

By routing the membrane tank return stream to the head of the aerobic zone in the configuration shown in Figure 6-1, the DO recycled from the membrane tank would be utilized in the aerobic zone. An internal return stream to the anaerobic/anoxic zones was provided by pumping mixed liquor from the end of the first stage of the aerobic zone. Because the DO level at the end of the first stage of the aerobic zone would be equal to the DO setpoint, which is less than the 4 – 5 mg/L in the membrane tank return stream, this internal pumping strategy would result in much less oxygen being returned to the anaerobic and anoxic zones. This configuration resulted in model-predicted effluent TP concentrations that were much less sensitive to changes in recycle flow rates.

To provide enhanced biological nitrogen removal in the future, modelling results indicated that this could be accomplished by moving the suction of the internal recycle stream which discharges to an anaerobic/anoxic zones from its current location, at the end of the first stage of the aerobic zone, to the end of the aerobic zone, where nitrate concentrations would be highest. This would, however, require a de-rating of the MBR treatment system.

6.4.3 Optimum Operation of an SBR for COD and Nitrogen Removal

The following case study is based on information presented in Andres *et al.* (2006).

Background and Objectives

The sequencing batch reactor (SBR) plant investigated as part of this study consists of four SBR units operating in parallel, treating wastewater from a commercial slaughterhouse operation. The effluent design objectives were 15 mg/L, 20 mg/L, and 3 mg/L for 5-day carbonaceous biochemical oxygen demand ($cBOD_5$), TSS, and TAN, respectively. In addition, seasonal effluent nitrate objectives ranged from 5 to 15 mg/L.

The existing SBR treatment system was incapable of meeting the effluent objectives. The purpose of the study was to utilize biological process modelling to determine if the duration of the SBR treatment cycles (fill, react, settle, decant) could be optimized such that effluent could meet the treatment objectives.

Model Development

GPS-X™, a biological process simulator program based on the ASM1 model, was used to develop a model of the existing treatment system. Default values were used for all kinetic and stoichiometric parameters.

SBR Cycle Optimization

A number of simulations were run to determine the optimal cycle time durations, with a total cycle time of 360 minutes. A sensitivity analysis protocol was utilized, whereby the duration of a cycle was increased and decreased, and the impact on model-predicted effluent quality was assessed.

The sensitivity analysis indicated that a longer settling cycle time was required to reduce effluent TSS concentrations, and that the existing aeration time was longer than required for adequate chemical oxygen demand (COD) removal and nitrification.

Finally, it was determined that an anoxic phase, after the aeration phase, could be utilized to promote denitrification, reducing effluent nitrate concentrations.

Optimal cycle time durations were developed for various treatment conditions, namely combinations of temperatures (i.e. representing summer, winter, and average) and flows (average and peak), for a total of six scenarios.

It was determined that the existing treatment system was capable of meeting the effluent objectives, with the exception of effluent nitrate at summer temperatures and peak flows, by varying the duration of the treatment cycles.

6.4.4 Use of Modelling to Evaluate Plant Capacity Under Increased Loading from an Industrial Source

The following case study is based on information presented in Andres *et al*. (2008).

Background and Objectives

The Galt WWTP is a conventional activated sludge process with tertiary filters, anaerobic sludge digestion and ultraviolet disinfection having a rated design capacity of 56,800 m^3/d (15.0 mgd). At the time of the study, the average raw influent flow was approximately 64% of the rated design capacity (36,000 m^3/d or 9.5 mgd).

The main objective of this project was to investigate the capacity of the plant under various loading scenarios and determine process requirements to achieve the desired effluent quality. The additional load sources included an industrial waste stream, dewatered centrate from an onsite sludge dewatering facility, and biosolids from other municipal facilities within the Regional Municipality of Waterloo.

Model Development and Calibration

A calibrated GPS-X^{TM} model of the existing treatment system was developed using existing plant operational data. The activated sludge system was modelled using the ASM2d biological model.

Currently only a portion of the industrial stream (approximately 20%) is directed to the Galt WWTP. Simulation scenarios with 40% to 100% of the industrial load being directed to Galt were investigated and the plant could meet all effluent criteria while treating 100% of the industrial load.

Once the plant capacity to handle the additional industrial load had been established, further simulation analysis was completed to investigate the capacity of the plant to dewater additional biosolids from other facilities within the Region. Dynamic simulation analysis was used to investigate different centrifuge operating schedules to minimize the sidestream impact on the liquid train capacity. At the optimal operating schedule, the liquid train could only handle 50% more dewatering centrate than the baseline without exceeding a maximum target MLSS concentration of 3,500 mg/L.

Two alternatives were identified to increase the capacity of the Galt WWTP liquid train to handle additional biosolids and asociated dewatering centrate loads:

- Construct an equalization tank to equalize the centrate return stream; and/or

- Reduce the percentage of the industrial flow that is directed to the Galt WWTP.

Dynamic modelling was used to quantify the impact of centrate equalization on the plant capacity. The required MLSS concentrations to maintain an effluent TAN concentration below 2.0 mg/L with and without centrate equalization is shown below in Figure 6-2.

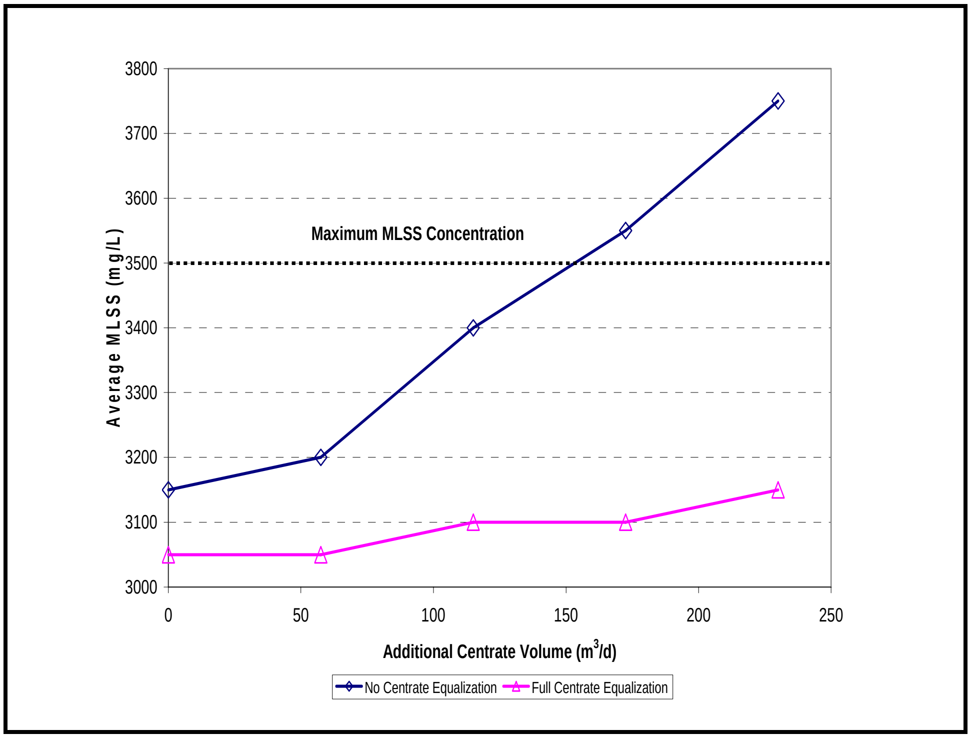

Figure 6-2 - Effect of Centrate Equalization on Required Operating MLSS Concentration

(From Andres *et al*., 2008)

Treating dewatering centrate in the activated sludge process is expected to be the most cost effective method of handling this internal side stream. Dynamic simulation analysis showed that equalization of the centrate stream will allow the plant to treat larger amounts of centrate with increased industrial waste contributions.

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CHAPTER 7

OPTIMIZATION TO MITIGATE EXTRANEOUS FLOW IMPACTS

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OPTIMIZATION TO MITIGATE EXTRANEOUS FLOW IMPACTS

7.1 TYPICAL SEWAGE CHARACTERISTICS AND FLOW PATTERNS

Flow to sewage treatment plants is made up of sewage discharged from residential, commercial, institutional, and industrial sources, plus sources of extraneous flows. Climate change is causing more intense, short duration storms resulting in high peak flows entering the sewage collection system and the sewage treatment plant as extraneous flow. There are a number of methods that can be implemented to mitigate the impacts of extraneous flows on sewage treatment plants. This chapter will focus primarily on mitigation of extraneous flows at the sewage treatment plant rather than within the collection system.

Sewage characteristics and flow patterns vary greatly between plants, and for this reason, whenever possible, sewage characteristics and flow patterns should be established based on collected flow data and sampling during dry and wet weather conditions (MOE, 2008). Sewage strength usually correlates with the volume of flow per person: with higher per capita flows diluting the concentrations of the constituents. Typical raw sewage composition is presented in Section 9.2.2.

Excluding extraneous sources of flow, a daily pattern for both sewage flow and composition is normally evident based on typical water usage patterns. As presented in Figure 7-1, the biochemical oxygen demand (BOD) concentration and flow pattern match consistently throughout the day with the peak diurnal flows and BOD peak loadings occurring in the morning and evening.

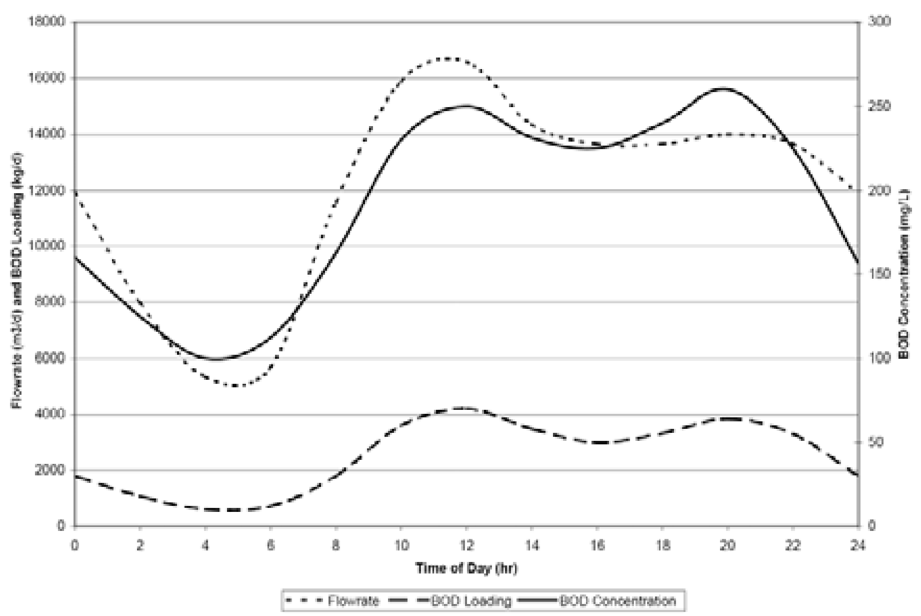

Figure 7-1 – Daily Sewage Flow and BOD Pattern

(Adapted from Metcalf & Eddy, 2003)

7.2 SOURCES OF EXTRANEOUS FLOW

Extraneous flow, often called infiltration/inflow (I/I), is clean groundwater or stormwater that enters the sanitary sewer system and is conveyed to the STP. There are several types of extraneous flows as outlined in Table 7-1.

Table 7-1 - Types of Extraneous Flow Contributions

(Adapted from Metcalf & Eddy, 2003)

Type of Extraneous Flow	Description of Extraneous Flow
Infiltration	When the groundwater table is high, water can enter collection system as a result of broken piping, manhole walls, piping connections or joints.
Steady Inflow	Continuously observed water entering collection systems as a result of drains connected in cellars, foundations, springs, or swampy areas. Steady inflow is measured along with infiltration.
Direct Inflow	Water entering collection systems as a result of a direct connection to the sanitary sewer including combined sewers collection systems, eaves trough drains, yard drains, or cross connections between storm drains and catch basins.

Table 7-1 – Types of Extraneous Flow Contributions (continued)

(Adapted from Metcalf & Eddy, 2003)

Type of Extraneous Flow	Description of Extraneous Flow
Total Inflow	Water entering collection systems as a result of all direct connections to the sanitary sewers, overflow discharged upstream of the treatment plant, or pumping station bypasses.
Delayed Inflow	Water entering collection systems that may drain several days following wet weather events from sump pumps or surface manholes that slowly moves into the collection system.

7.3 ESTIMATING EXTRANEOUS FLOW CONTRIBUTION

Estimating the extraneous flow contributions to a sewage treatment plant requires extensive data collection during dry weather flow conditions when the groundwater table is believed to be low as well as during and following wet weather events when the groundwater table is believed to be high. Once wet and dry weather flows into the sewage treatment plant are well defined, it is possible to plot and compare the flows to estimate the contributions from the various types of extraneous flow. Figure 7-2 identifies extraneous flow contributions by plotting dry-weather flow patterns (Figure 7-1) and wet-weather flow patterns on the same graph.

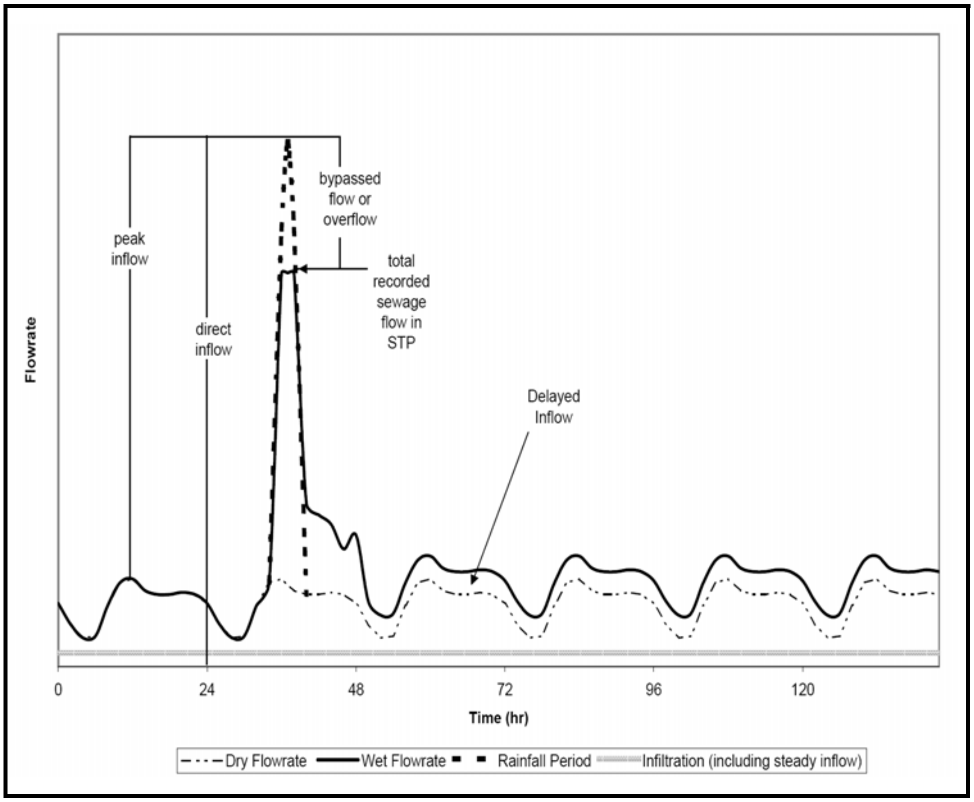

Figure 7-2 – Interpretation of Wet and Dry Weather Flow Data to Identify I/I

(Adapted from Metcalf & Eddy, 2003)

As shown in Figure 7-2, comparing dry weather flow to wet weather flow allows an estimate of the contribution from various types of extraneous flow to be made.

Based on collection system length, the typical range of extraneous flow as a result of infiltration is between 0.01 and 1.0 $(m^3/d)\cdot mm\cdot km$ with the diameter of the collection sewers in millimeters and length of the sewer system in kilometers (Metcalf & Eddy, 2003). Based on the area served by the collection system, a range of between 0.2 and 28 $m^3/ha\cdot d$ is typical (Metcalf & Eddy, 2003).

Infiltration and inflow volumes and patterns are specific to a sewage treatment plant and collection system due to a number of factors including:

- Relative height of groundwater table to collection system;
- Type of soil;
- Area served by the collection system;
- Construction material of collection system;
- Quality of construction of the collection system;

- Age of the collection system; and
- Frequency, duration and intensity of wet weather events.

Further information on estimating extraneous flow contributions can be found in FMC and NRC (2003) and Metcalf & Eddy (2003).

7.4 IMPACT OF EXTRANEOUS FLOW ON SEWAGE TREATMENT PLANT OPERATION

The impact of extraneous flow on the operation of sewage treatment plants varies depending on the extent and type of extraneous flows into the STP. Table 7-2 outlines the expected impacts of extraneous flow on individual unit processes. Extraneous flow will impact those unit processes that are sensitive to hydraulic loading; however, the greatest impact is the disturbance on the solids separation unit processes such as primary and secondary clarifiers and tertiary treatment units.

7.5 ATTENTUATION OF WET WEATHER PEAK FLOWS

There are a number of methods that can be implemented to mitigate the impact of wet weather peak flow either by reducing the volume of I/I into the system or by managing the rate of flow through the sewage treatment plant.

7.5.1 Infiltration and Inflow Reduction

Mitigation of I/I involves developing an understanding of the sources of extraneous flow within a collection system. Once the sources of I/I are determined and prioritized based on level of impact, a systematic approach should be developed to disconnect, repair or eliminate the connections to the collection system. Depending on the specific system this can be a time consuming and expensive process.

More information on reducing infiltration and inflow can be found in FCM and NRC (2003).

Table 7-2 - Impact of Extraneous Flows on Sewage Treatment Unit Processes

Unit Process	Impact of Extraneous Flow
Preliminary Treatment	• Hydraulic overloading, leading to decreased removal efficiency • Potential for blinding of screens, overloading of grit chambers as the scouring of sewers due to extraneous flows can result in high levels of debris and grit being conveyed to the STP (Chapter 10)
Primary Clarification	• Hydraulic overloading, leading to decreased removal efficiency • Hydraulic overloading potentially causing solids washout to downstream processes (Chapter 11)
Biological Treatment	• Hydraulic overloading potentially causing biomass loss from suspended growth processes (Chapter 12) • Biomass washout will result in a decrease in treatment efficiency • Fixed film processes are less susceptible to biomass loss due to hydraulic overloading
Secondary Clarification	• Hydraulic and solids loading rate overloading due to biomass and solids washout from upstream processes resulting in poor settling and the potential for solids carry-over which can impact downstream processes and effluent quality (Chapter 14) • Solids carry-over out of the secondary clarifier can impact the quality of return activated sludge to the suspended growth biological process and therefore further impact treatment capacity and efficiency
Tertiary Treatment	• Hydraulically overloading tertiary treatment processes can result in decreased process efficiency • For some filtration processes, solids washout from upstream processes can blind the filters resulting in a decrease in hydraulic capacity (Chapter 15)
Disinfection	• For UV disinfection, the presence of higher turbidity levels can result in decreased efficiency (Chapter 18) • For chlorination and UV disinfection, reduced contact time can lead to reduced disinfection efficiency

7.5.2 Flow Equalization

Flow equalization can be utilized to minimize both fluctuations in daily flow patterns as well as the impacts of extraneous flows, especially during wet weather events. Flow equalization involves the collection of all or a portion of the flow to a sewage treatment plant prior to treatment, followed by controlled release to dampen the impacts of hydraulic and organic loading fluctuations (MOE, 2008; Metcalf & Eddy, 2003). Flow equalization can improve the treatment efficiency and energy usage within a sewage treatment plant by maintaining a consistent hydraulic flow through the treatment processes (MOE, 2008). Flow equalization can take place within the collection system or at the STP.

Within the collection system, optimizing the pumping rate, and potentially the wet well level set-points at sewage pumping stations can provide additional flow equalization. Optimization of sewage pumping stations is discussed in Chapter 8.

At sewage treatment plants, flow equalization usually takes place downstream of preliminary treatment to ensure grit and debris is removed (MOE, 2008). Flow equalization at sewage treatment plants can be accomplished in a number of ways including:

- Utilizing existing aeration tanks, sedimentation tanks, digesters, lagoons, or other process tanks which are not currently in use;

- Full-flow or side-stream retention or treatment basins which are able to store or treat extraneous flow above the STP's treatment capacity; or

- In-line treatment or storage units able to dampen the impact of flow variations.

Further information on implementation of flow equalization can be found in MOE (2008) and Metcalf & Eddy (2003).

7.6 OPTIMIZATION OF WET WEATHER FLOW TREATMENT CAPACITY

This section focuses on ways to optimize the wet weather treatment capacity of a STP without requiring plant expansion or the implementation of separate extraneous flow treatment processes. It should be noted that for some sewer systems that have regular combined sewer overflow (CSO) incidents, the installation of a separate wet weather treatment facility should be investigated. Information on CSO treatment can be found in MOE (2008) and NWRI *et al.* (2005).

Simulation modelling can be used to gain insight into the impact that wet weather flow events can have on individual sewage treatment process as well as on the overall plant treatment efficiency. Modelling can be a useful tool to determine the processes that are most impacted by wet weather flow as well as to simulate any optimization measures before implementation. More information on modelling and simulation is provided in Chapter 6.

7.6.1 Step-Feed Operation

Wet weather flow events most significantly affect secondary clarifiers by increasing the solids loading rate to beyond the clarifier design capacity. One method utilized to decrease the solids loading to the secondary clarifier during wet weather flows to prevent biomass washout is implementation of step-feed to the upstream aeration basins.

Step-feeding entails controlling the influent flow into the aeration basins by distributing the flow between at least two points along the length of the reactor. As shown in Figure 7-3, it is possible to change plug flow reactors to step-feed by continuing return activated sludge (RAS) flow to the head of the aeration basin but changing the point at which the primary effluent enters from the head of the aeration basin to more than one point along the length of the basin (Marten *et al*., 2004). By staggering the addition of the primary effluent, the MLSS concentration at the head of the aeration basin is equal to the RAS solids concentration and is further diluted at each addition point of primary effluent. This results in a decrease in the solids loading rate to the secondary clarifiers (Marten *et al.*, 2004).

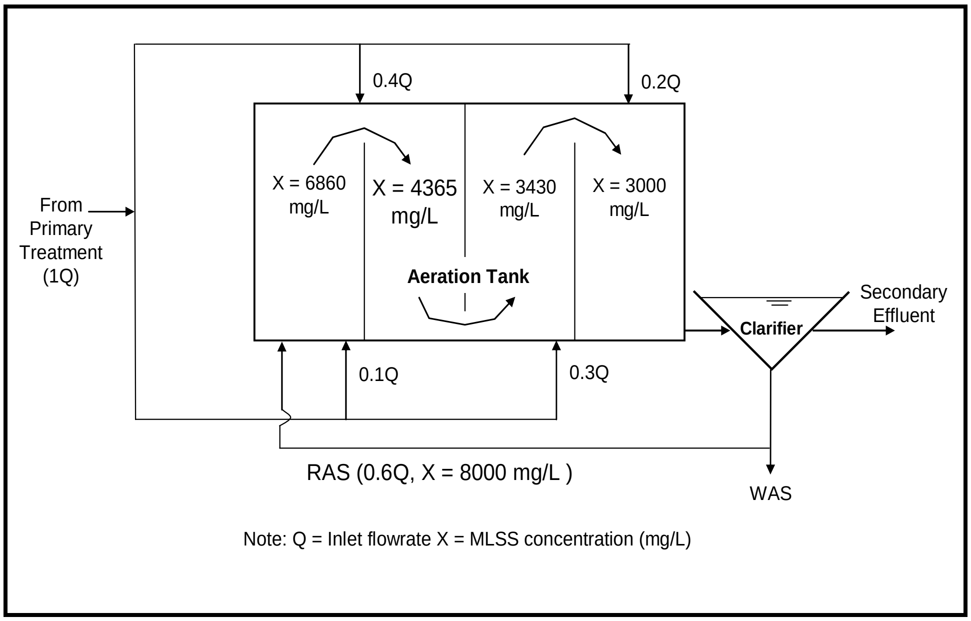

Figure 7-3 – Example of a Step-Feed Activated Sludge Process

(Adapted from Metcalf & Eddy, 2003)

A key requirement of implementing step-feed at sewage treatment plants is the ability to separate the RAS and primary effluent flows into the aeration basin. If this is not possible, alternatives to step-feed must be implemented.

Further information on step-feed to control wet weather flows can be found in Thompson *et al.* (1992).

7.6.2 Solids Storage within Aeration Basins During Peak Flow Events

When step-feed (Section 7.6.1) cannot be implemented and there are several aeration basins at a plant, an alternative strategy to minimize the impact of peak flow events is to convert a portion of the aeration basins to temporary solids storage basins during peak flow events.

During dry weather, the aeration basins would operate normally. As shown in Figure 7-4, during wet weather peak flow events, a portion of the aeration basins would be converted into clarifiers (without solids removal) by turning the air off (Marten *et al*., 2004). The effluent from these solids storage aeration basins has a comparatively low suspended solids concentration and dilutes the overall mixed liquor concentration flowing to the secondary clarifier. When the solids concentration flowing from the solids storage aeration basins increases (as indicated by a solids-density meter), the inlet gates to the basins are closed and the aeration turned back on to ensure that the solids do not turn anaerobic. Following the peak flow event, the inlet gates to the chambers are partially opened to allow fresh mixed liquor into the aeration basins and the concentrated solids slowly flow to the secondary clarifier. As the solids content of the solids storage aeration basin returns to normal levels the inlet gates are opened fully until the next peak flow event.

Further information on converting aeration basins to temporary solids storage tanks during peak flows can be found in Marten *et al*. (2004).

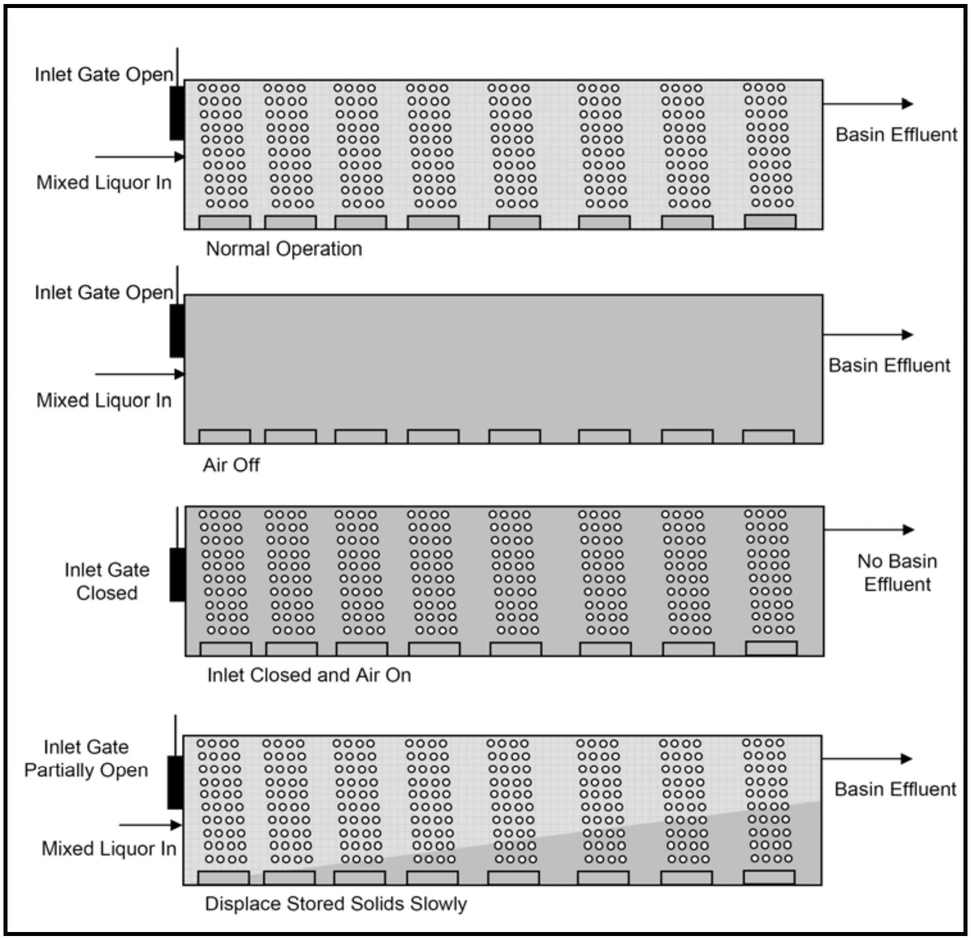

Figure 7-4 - Solids Storage Aeration Basins Sequence of Operation

(Adapted from Marten *et al*., 2004)

7.6.3 Chemically Enhanced Sedimentation

Chemically enhanced primary treatment (CEPT, Section 11.2) and chemically enhanced secondary clarification (Section 14.3.3) can increase the hydraulic capacity of a clarifier while maintaining clarifier performance and effluent quality.

Chemically enhanced sedimentation involves chemical coagulation and flocculation to enhance the performance of the clarifiers by increasing the fraction of settleable solids, improving the settleability of the solids, and increasing the settling rate resulting in increased capacity to treat wet weather flows.

CEPT is discussed in Chapter 11 and optimization of the secondary clarifier is discussed in Chapter 14.

7.7 CASE HISTORIES

7.7.1 Step-Feed Control at Dundas Water Pollution Control Plant to Control Wet Weather Flows

The following case study is based on information presented in Thompson *et al.* (1992).

A demonstration project was undertaken at Dundas Water Pollution Control Plant (WPCP) in 1990 to develop a control strategy for CSOs. The Dundas WPCP has two separate plants (A and B) and at the time of the demonstration project had a total design flow of 18,000 m^3/d. The step-feed demonstration was carried out in Plant A which allowed for side-by-side comparisons. In addition, this configuration allowed for the entire plant flow to be diverted to Plant A to simulate storm flow conditions.

Step-feed was initiated by manipulating two gates that controlled the primary effluent flow directed to Plant A's two completely mixed aeration tanks in series. Plant A was monitored while operating with and without step-feed for several months. The effluent suspended solids and BOD_5 concentrations were below 10 mg/L during both operating conditions. During conventional operation, the plant was able to achieve complete nitrification; however, during step-feed, there was some ammonia bleed-through although the TKN concentration remained below 2 mg/L.

During the demonstration, the plant experienced several wet weather events which resulted in flows as high as three time the peak dry weather flow. During non step-feed operation, the influent (potential bypass) had a low BOD_5 but high suspended solids concentration of 200 mg/L showing that the primary settlers were not effective at consistently removing suspended solids during peak flows.

During step-feed operation, there was an initial transfer of solids from the aeration basin to the secondary clarifier increasing the sludge blanket height. Over time, as the step-feed lowered the solids loading to the clarifier, there was a reduction in the sludge blanket height to a stable operating level and the effluent suspended solids concentration remained below 20 mg/L.

Overall, the demonstration project confirmed the effectiveness of step-feed as a means of avoiding solids washout during peak flow events.

7.7.2 Wet Weather Flow Treatment Strategy at Ashbridges Bay Treatment Plant (Toronto)

The following case study is based on information presented in Zegers *et al.* (2009).

The Ashbridges Bay Treatment Plant is Toronto's largest STP and treats flows from both separate and combined sewer systems with an average daily capacity of 0.818 million m^3/d. During wet weather flow conditions, the peak instantaneous flow is over four times the average daily flow and the primary effluent flow that bypasses the secondary treatment process stage is disinfected and then discharged to Lake Ontario.

Secondary bypass occurrences represent a small fraction of the total sewage flow (1.4 percent) but are a considerable fraction of the plant's effluent pollutant load. To improve the quality of secondary treatment bypasses, a wet weather flow strategy was developed that uses the existing infrastructure including:

- Split flow treatment;
- CEPT; and
- Potentially high-rate treatment.

The first component of the strategy to be implemented is the flow splitting as it was integrated into forth-coming construction plans. The flow splitting will allow a portion of the screened and degritted flow to bypass the primary sedimentation tanks and be sent directly to the aeration basins which will hopefully reduce overloading and solid washout from the primary clarifiers and improve secondary bypass quality.

Flow splitting would be initiated when the rated peak flow capacity is exceeded (2.4 million m^3/d) at which point a secondary bypass would already be underway. By splitting the flow at this point, the first flush of the wet weather flow with the highest solids load would be treated by the primary process, and then during flow splitting, the sewage would be more dilute and therefore not overload the secondary process. A new gate is to be installed into the aeration tank influent chamber to prevent bypass of the screened and degritted raw sewage during split flow operation.

The second component of the strategy to be implemented is CEPT which is intended to increase primary treatment capacity from 0.966 to 1.3 million m^3/d. Chemical dosing to the primary clarifiers would only take place during the high flow conditions with a rapid mixing and flocculation chemical system. Jar testing followed by pilot scale testing will determine the appropriate chemicals and dosages. Jar testing conducted in 2008 indicated that dosing a combination of 5 mg/L of ferric chloride (as Fe), 0.5 mg/L of polyaluminum chloride (as Al_2O_3) and 0.4 mg/L of anionic polymer achieved 81 percent TSS removal and 73 percent $CBOD_5$ removal.

The Ashbridges Bay Treatment Plant's wet weather treatment strategy is still being implemented with pilot scale testing of CEPT and the potential to install high rate treatment units to be investigated following the City of Toronto's development of a city-wide CSO abatement plan.

7.7.3 Secondary Treatment Optimization to Maximize Wet Weather Capacity in Wisconsin

The following case study is based on information presented in Marten *et al.* (2004).

The Jones Island Wastewater Treatment Plant located in Milwaukee, Wisconsin is the oldest activated sludge plant in the United States with a nominal peak capacity of 1.2 million m^3/d of sewage from separate and combined sewers. The Milwaukee Metropolitan Sewerage District also has more the 31 km of deep tunnels located 91 m underground, able to store 1.5 million m^3 of wet weather flow for later treatment. The inline storage has been in place since 1994 and since then the overflow events have dropped from over 50 to an average of 3 per year. That being said, the District wished to have no overflow events per year. From studies conducted it was found that the secondary clarifier capacity was the limiting process with a maximum capacity of 0.946 million m^3/d, which was below the plant's nominal peak capacity. As a result of this assessment, a study was undertaken to modify the activated sludge process to lower the solids loading to the secondary clarifiers under peak flow conditions and improve clarifier performance.

Due to the design of the aeration basins, they could not be operated in a step-feed mode. An alternative solution of modifying 20 percent of the aeration basins to solids storage aeration basins during peak flow conditions was implemented (Section 7.6.2) which required the following system changes:

- Diffusers were switched to fine bubble membranes from fine bubble ceramic plate diffusers to allow the aeration system to be turned on and off;

- Addition of electric actuators to the inlet gates;

- Altering the outlets of the basin to allow the effluent from storage and non-storage basins to mix upstream of the clarifiers; and

- Modification of the control system to automatically switch between peak flow and normal operating conditions.

This change in operation was expected to reduce the MLSS concentration of secondary clarifier influent from 2,200 mg/L under normal conditions to 1,520 mg/L during peak flow conditions, which would result in a 30 percent decrease in solids loading rate and give the plant the ability to handle 40 percent more flow.

In addition to the above-noted measures, improvements to the clarifiers were undertaken. Jones Island Wastewater Treatment Plant has a total of 33 secondary clarifiers in three groupings. The oldest 10 clarifiers, which make up 50 percent of the clarifier surface area, were reviewed to determine methods to improve clarifier performance as they were capable of handling only 30 to 35 percent of the peak flow.

These clarifiers had poor inlet design which resulted in short-circuiting and poor flocculation as well as areas of high surface overflow rate. These clarifiers were double-square in construction with filleted corners and circular rapid sludge withdrawal mechanisms. The inlet design consists of rectangular openings at the top of two 650 mm diameter pipes placed at the centre of each half-clarifier. The pipes brought mixed liquor into the clarifiers at a high velocity of 0.3 to 0.5 m/s and the momentum resulted in the majority of the solids deposited to the west half of each clarifier. To improve the performance of the clarifiers, two clarifiers were modified and underwent long term monitoring. The modifications included installation of Stamford baffles around the perimeter of the weir troughs and blocking the corner weirs for both clarifiers. One of the clarifier's inlets was also retrofitted with flocculating energy dissipating well arrangements (FEDWAs) which are designed to promote better mixed liquor flocculation and prevent hydraulic density currents. More information on clarifier baffling, stress testing and hydraulic testing is provided in Chapter 14.

Initially dye and stress testing showed that both clarifiers dramatically improved in performance compared to the unmodified clarifiers with little difference between the two modified clarifiers. Longer term monitoring indicated that the clarifier with FEDWAs was less likely to have a rising sludge blanket during high flow conditions and also had a higher RAS concentration (2000 mg/L) compared to the other clarifier (1000 mg/L).

At the time of publication, numerous improvements were in place including modifications to:

- Two of six aeration basins that serve as high-flow solids storage aeration basins;

- Seven of ten clarifiers had Stamford baffles and blocked corner weir baffles; and

- One of the ten clarifiers had FEDWA inlets.

At that time, one large wet weather event had occurred and the temporary solids storage aeration basins were utilized. The MLSS in the activated sludge system decreased from 2,700 mg/L to 2,400 mg/L. Using solids flux models, it was determined that the secondary treatment process could treat more than 1.1 million m^3/d of flow for most of the storm event with the remaining directed to the tunnels. During this wet weather event and with a partially completed facility, the plant was able to handle a 10 percent increase in flow compared to the unmodified operation.

After the remaining four temporary solids storage aeration basins come online, nine more FEDWA inlets are installed, and five more clarifiers are retrofitted with Stamford baffles and blocked corner weirs, the goal of dependably restoring the 1.2 million m^3/d capacity will be realized.

7.8 REFERENCES

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CHAPTER 8

SEWAGE PUMPING STATION OPTIMIZATION

CHAPTER 8

SEWAGE PUMPING STATION OPTIMIZATION

8.1 IMPACT OF PUMPING STATION OPERATION ON STP PERFORMANCE

The purpose of sewage pumping is to allow for the conveyance of sewage through a STP over the range of expected conditions. Sewage pumping may be required at facilities where insufficient differences in ground elevation, site constraints, or rugged, uneven terrain prevent the flow of sewage through a STP by gravity.

Sewage pumping stations are often incorporated into the design of sewage collection systems. The operation of these pumping stations can impact flows to the downstream STP. In some instances, a raw sewage pumping station is located directly upstream of a STP and all sewage into the STP flows through the pumping station. In this case, the flows experienced through the STP are dictated by the operation of this raw sewage pumping station.

Depending on the STP layout and treatment processes, pumping may be required into the plant or as an intermediate step between unit processes. Where a pumping station is in operation at a STP, the flows going through the treatment processes are governed by the pumping station. In such instances, the duration and intensity of peak flows to downstream processes are controlled by the operation of the pumping station.

In a typical secondary STP, different types of pumping systems may also be present to serve various unit processes throughout the STP (RAS, WAS, primary sludge, raw sewage, effluent, etc.) This chapter will focus on raw sewage pumping stations (or intermediate steps between unit processes); however, many of the concepts discussed can also be applied to other pumping applications.

The two most common types of sewage pumping stations are the wet well/dry well and submersible pump stations (EPA, 2000). In a wet well/dry well pumping station, the pumps, valves and equipment are contained within a readily accessible pump room (dry well), separate from the wet well. In a submersible pump station, the pumps are not housed in a room separate from the wet well. Rather, as the name implies, the pumps are submerged in the wet well, while other appurtenances (valves, instrumentation) are housed in a separate room. Other types of pumping stations include suction lift (where the self-priming pumps are located above the water level) and screw lift (an Archimedean screw with a motor mounted above the water level) (MOE, 2008).

Symptoms and causes of common problems encountered with pumping stations are presented in Table 8-1.

Table 8-1 - Pumping Station - Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Hydraulic bottleneck at the pumping station/pumps.	• Overflow of upstream processes, tanks, channels, pipes or wet wells • Operating above firm capacity for extended periods	• Undersized pumps (Section 8.2.1 and Section 8.2.5) • No equalization (Section 8.2.2) • Clogged pumps (Section 8.2.4) • Undersized forcemain/piping
Frequent cycling of pump operation.	• Inconsistent flows resulting in alternating periods of flow and no-flow (and loading) to treatment processes (i.e. biological processes) • Settling of solids and grit in channels, pipes or tanks during no-flow or low flow conditions	• Oversized pumps (Section 8.2.1 and Section 8.2.5) • Pumps not equipped with variable frequency drives (VFDs) (Section 8.2.3)

8.2 OPTIMIZATION APPROACHES

8.2.1 Pump Selection and Sizing

Undersized pumps can result in hydraulic bottlenecks, causing sewage to back-up and to overflow upstream processes, tankage, pipes and wet wells. This can lead to bypassing of the pumping station wet well and the potential discharge of untreated or partially treated sewage to receiving waters.

To minimize the occurrence of sewage back-up and overflows, additional pumps can be installed if existing connections exist, or the pumps and motors can be replaced with higher capacity units.

The selection and sizing of pumps should be based on the minimum and maximum hourly flows to provide a steady flow to the downstream unit processes throughout the day. Multiple pumps should be provided, and sized such that the firm capacity is capable of handling at least the 10-year design peak hourly flow (MOE, 2008). Consideration should also be given to the impacts of peak design instantaneous flows on required pump sizes.

In small pumping stations, two pumps are typically installed, each sized to handle the peak design capacity of the pumping station. To optimize pump efficiency in pumping stations with high peak flows, consideration should be given to the installation of additional pumps to provide intermediate capacities to handle the typical daily flows (EPA, 2000).

Oversized pumps can operate in an on-off mode during low flow conditions, causing uneven flows or periods of no-flow to downstream unit processes. This can cause the settling of suspended solids and grit in downstream processes, pipes and tanks due to insufficient turbulence to maintain solids in suspension. The settling of grit can reduce effective tank volumes and operating capacity.

Where the pumping station discharges to a sewage treatment plant, the pumps must be able to operate over the entire range of flows to the plant. Problems with pump over-sizing are commonly encountered in new or newly expanded pumping stations where the pumps were sized to be capable of handling the expected peak flows at build-out, without consideration of the minimum hourly flows at present conditions. The installation of multiple, smaller capacity pumps, which operate according to wet well level can minimize the frequency of on-off cycling and provide more consistent flows to the STP throughout the day.

8.2.2 Equalization

Where pumps are undersized, the installation of an equalization tank can serve to buffer the peak flows and to provide storage for flows in excess of the pumping station capacity for conveyance once the peak flows have subsided. Sizing of an equalization tank should be based on the pumping station capacity, and the magnitude and duration of the peak flow.

Refer to Section 7.5.2 for additional information on flow equalization to mitigate the impacts of extraneous flows.

8.2.3 Variable Frequency Drives

Where pumping station configuration does not allow for the installation of multiple, lower capacity pumps in place of a single larger capacity pump, a variable frequency drive (VFD) may be installed on the existing pump(s).

The purpose of a VFD is to allow a degree of control over the output of the pump by controlling the current input to the motor. By varying the current to the motor, the speed of the motor and pump can be controlled. As opposed to throttling the output of the pump with control valves, adjusting the operating speed of the motor and pump reduces the output of the pump from the source and saves energy by optimizing the pump operation.

Installation of a VFD allows the pump to operate at different pump outputs to match varying flow conditions, optimizing pump operation by providing flexibility to operate over a range of flows. This effectively maintains some flow to downstream processes and minimizes the settling and accumulation of solids and grit in tanks and channels.

VFDs can also allow for soft starts and stops, minimizing hydraulic and mechanical stresses on system piping, channels and unit processes and equipment. Hydraulic stresses, often referred to as water-hammer, are the result of sudden increases in pressure, sending out shock waves and potentially damaging system components. Mechanical stresses refer to the mechanical wear that motors and pumps undergo as a result of frequent starts and stops.

Installation of VFDs can optimize energy usage by reducing the power going to the motor at lower flows, and reducing the frequency of energy intensive pump start cycles.

Most pumps are operated between 50 and 100 percent of the rated capacity. This is limited by the motor and equipment. Motors are typically cooled by a fan on the same drive as the motor, and the fan operates at the same speed as the motor. At low speeds, the fan does not rotate rapidly enough to provide sufficient airflow to cool the motor, resulting in increased mechanical stress and rapid wear. Equipment manuals or suppliers should be referred to in order to determine the optimum operating range for the existing pump and motor assembly.

With multiple VFD-equipped pumps, the pumps can be operated in a load sharing mode - the simultaneous operation of both pumps at the same speeds, or non-load sharing - the designation of a lead and lag pump to operate in sequence based on the control set point (typically wet well level). In non-load sharing operation, where the pumps are equally sized, the lead pump speed must be decreased to match the lag pump speed when it comes online (WEF, 1997).

A combination of fixed speed and VFD-equipped pumps has been used in many large pumping stations to obtain the benefits of both types of pumps. This introduces more complexity into the control strategy due to flow rate discontinuities or gaps. These gaps may result in uneven flow and surges to downstream unit processes and harmful pump cycling (WEF, 1997).

Minimizing these flow rate gaps is often achieved by the combination of large, VFD-equipped lead pumps and smaller, fixed speed lag pumps. Selecting operating setpoints that overlap previous operating setpoints further reduces flow rate gaps.

Control Strategies

There are several control strategies typically employed at pumping stations:

- Level setpoint control;
- Level band control; and
- Discharge flow rate control.

In level setpoint control, pump station operation is dictated by the liquid level(s) in the wet well. Specific setpoints are set based on different water levels within the wet well, and pump start sequence and operation are based on the set points. Maintaining control too tightly can reduce the effectiveness of wet well storage in dampening flows to the plant. The drive speed of the pumps may also vary wildly in an attempt to maintain wet well level with varying flows.

Level band control is a variation on level setpoint control where pump operation steps are based on wet well level ranges rather than setpoints based on distinct wet well levels. By setting the steps to operate over overlapping ranges, the discharge flow rate is smoothed. This has the benefit of dampening peak flows to the STP.

Where an equalization tank is present with large wet wells, level control may not be as important as in pumping stations with smaller wet wells. In such cases, discharge flow control can be used rather than level control. This control strategy can optimize the flows to downstream processes, ensuring even, consistent flows and dampening peak flow events to the STP (WEF, 1997).

8.2.4 Pump Clogging

Pumps, particularly in the case of combined sewers, may be prone to clogging due to the presence of coarse debris in the raw sewage stream. If not removed upstream of the pumps, the coarse material may result in clogged pumps and/or damage to the pumps and/or motors. Pump clogging may also result in overflow of upstream channels and/or unit processes.

Consideration should be given to the installation of coarse screens upstream of the pumps to remove coarse material and debris from the sewage stream prior to pumping, minimizing pump downtime. For details regarding the design and selection of screening devices, reference should be made to MOE (2008).

Installation of pumps capable of passing objects of up to 80mm in diameter should help reduce the frequency of clogging and minimize damage to the pumps and motors as a result of clogging (MOE, 2008).

8.2.5 Impeller Modification

Where a pump is undersized or oversized, or where downstream hydraulic conditions have changed, impeller replacement or modification can potentially eliminate the need for pump replacement.

Modifying or replacing the impeller in a centrifugal pump shifts the pump’s operating curve, effectively changing the operating point of the pump. In addition to potentially avoiding costs associated with pump replacement, impeller modification or replacement can allow for more efficient operation of the pump, reducing operating costs.

Depending on the size of the pump volute and existing impeller, it may not always be possible to replace the impeller with one of a larger or smaller size. In such cases, if a smaller impeller is required for an oversized pump, the impeller can be trimmed to reduce its size. Conversely, if a larger impeller is needed, total pump replacement may be required.

The selection or modification of a pump impeller is based on the size of the pump, the system head curve, pump configuration, pump power and required capacity. Pump suppliers should be consulted when considering modification or replacement of an impeller, to ensure that the new or modified impeller will not negatively impact pump performance.

8.3 CASE HISTORY

8.3.1 Reservoir Avenue Pump Station Optimization

The following case study is based on information presented in U.S. Department of Energy (2005).

Background and Objectives

The Town of Trumbull is located north of Bridgeport in Connecticut and has a population of 32,000. Sewage from the Town is treated in the nearby city of Bridgeport, and is conveyed to the STP via ten sewage pumping stations, with a combined capacity of 12,492 m^3/d (3.3 MUSGD).

The Reservoir Avenue Pump Station was constructed in 1971 and consists of two variable speed pumps handling approximately 1,287 m^3/d (0.34 MUSGD) of raw sewage. The two pumps are operated in a lead-lag pump configuration with the lead pump operating continuously during typical flow conditions and the second pump coming online during high flows. Pump operation is controlled by bubbler level control. Historically, the pumps rarely operated for durations in excess of five minutes.

The goal of the study was to optimize the energy consumption of the sewage pumping station by identifying areas for potential energy savings, and by implementing solutions to reduce the energy usage.

Optimization Methodology

A systems approach was utilized for optimization of the sewage pumping station. Total system performance was examined to identify areas for energy savings, including pump system operation and frictional losses in piping. Where identified, modifications to the existing pumping station were implemented.

Total System Performance

Initial testing had identified the following areas for optimization:

- The existing pumps typically operated at 4,633 m^3/d (850 USGPM), resulting in on-off cycling at typical flows. As a result, the pumps and motors were subject to increased wear and were prone to mechanical breakdown. Further, the high pump output resulted in large frictional losses in system piping, increasing energy demand;

- The pump speed control system was not functioning correctly. As a result, the pumps and motors had been operating inefficiently at constant reduced speeds;

- As a result of the ineffective speed control system, two circulating cooling water pumps for the motors were constantly in operation; and

- Other sources of energy consumption were identified in the level control system, which was equipped with two continuously operating compressors, and in the pump station lighting, which, as a result of a broken automatic light switch, were constantly on.

Pumping Station Modifications

The following modifications to the Reservoir Avenue Pump Station were implemented following the results of the total system performance examination:

- One 2,453 m^3/d (450 USGPM) capacity pump was installed as the new lead pump to reduce the frequency of on-off cycles, and to reduce the frictional losses in system piping. The existing pumps were retained to handle peak flow events;

- The pump speed control system was eliminated entirely. As a result, the operating speed of the existing motors was increased, requiring modifications to the pump impellers to compensate;

- As a result of the elimination of the speed control system, the cooling system for the existing motors was eliminated as well;

- The bubbler level control system was replaced with a float switch control system; and

- The automatic light switch for the pumping station was repaired.

Results

The following were the results of the optimization program for the Reservoir Avenue Pump Station:

- The installation of the new, smaller capacity lead pump resulted in an energy savings of 17,643 kWh. Additionally, the reduced on-off operation has also reduced the frequency of maintenance associated with pump and motor wear and breakdown;

- Elimination of the bubbler level control and motor cooling systems has resulted in a savings of 7,300 kWh/yr and 1,752 kWh/yr, respectively;

- Repairs to the automatic light switch at the pump station have reduced lighting energy consumption from 5,256 kWh to 78 kWh; and

- In addition to the energy savings, the reduced maintenance requirements for the station as a whole would reduce labour and associated costs.

As a result of the optimization of the Reservoir Avenue Pump Station, an overall energy savings of 31,875 kWh was realized. This is equivalent to a reduction in energy consumption of approximately 44 percent, or approximately $2,600/yr. Based on a project implementation cost of approximately $12,000, the payback period is 4.6 years.

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CHAPTER 9

FLOW METERING AND SAMPLING

CHAPTER 9

FLOW METERING AND SAMPLING

9.1 OVERVIEW OF FLOW METERING

9.1.1 Purpose of Flow Metering and Types of Flow Meters

The purpose of flow metering is to accurately measure and record the volume of a fluid (liquid, including suspensions such as sludge, or gas) passing through a conduit over a given period of time. At sewage treatment facilities, flow metering is commonly used to measure the flow rate of various streams in both the liquid train and sludge processing train for the purposes of process control and, in some instances, billing.

Accurate flow data is essential when undertaking process evaluation or optimization activities at a STP. Accurate flow data will provide insight into the actual operating conditions within unit processes, and form a basis with which to evaluate process performance.

Flow metering data can be provided in terms of totalized flows (the total volume of flow over a specified time period, generally 24 hours), instantaneous flows recorded at a given instant in time (such as maximum or minimum flows during the day), or continuous flow recording (generally a time-series of instantaneous flows captured at a specified time interval, such as 1 minute). The type of flow data required depends on the process stream being monitored and measured.

Flow meters can be divided into two main categories: open channel flow metering devices, and closed conduit flow metering devices. Both types of flow meters are widely used in sewage treatment facilities.

Figure 9-1 - Example Installation of a Rectangular Weir

Open Channel Flow Meters

Open channel flow meters generally utilize a primary measuring device, such as a weir or flume. These are hydraulic structures that are placed into the channel to change the liquid level in or near the structure. The liquid level (head) varies in proportion to the rate of flow within the channel according to a mathematical relationship specific to the type and dimensions of the primary measuring device. A secondary measuring device is used to measure the liquid level(s) that are input into the mathematical relationship. Examples of secondary measuring devices include ultrasonic level sensors, bubblers, and pressure transducers.

One specific type of open channel flow meter, the area-velocity (AV) flow meter, does not utilize a hydraulic structure to alter the level within the channel; rather, both the velocity of flow and liquid level within the channel are measured and used to calculate the flow rate. These types of flow meters consist of two sensors: a Doppler sensor, to measure the velocity of flow within the channel, and a level sensor, which are used together with the cross-sectional dimensions of the channel to calculate an area of flow. The velocity and area are used to calculate the volumetric flow rate.

Closed Conduit Flow Meters

The most commonly used closed conduit flow meters in STP applications include magnetic flow meters (magmeters), Venturi meters and Doppler meters. Closed conduit flow meters use varying methods for determining flow rate. Magmeters create a magnetic field perpendicular to the direction of flow, and a voltage is induced that is proportional to the flow rate. Venturi meters utilize a change in cross-sectional area to induce a pressure differential across the venturi. The pressure differential is a function of flow rate. Doppler meters measure the velocity of particulate matter in the liquid stream, and hence the velocity of the fluid flow. The velocity along with the dimensions of the closed conduit, are used to determine the flow rate.

Table 9-1 presents a summary of various types of flow meters commonly used at sewage treatment facilities.

Table 9-1 – Flow Metering – Types of Flow Meters Commonly Utilized at Sewage Treatment Facilities

Category	Common Types of Flow Meters	Common Applications
Open Channel	• Flumes o Parshall o Palmer-Bowlus o Leopold-Lagco • Weirs o Rectangular o Proportional o Trapezoidal (Cipolletti) o V-notch (triangular) • Area-velocity	• Plant influent / effluent flows • Bypass flows • Flow through any open channel
Closed Conduit	• Magnetic (mag) meters • Venturi meters • Doppler meters	• RAS / WAS flow metering • Influent / effluent forcemains • Flow through any closed conduit

9.1.2 Evaluating Flow Data

Continuity testing can be used to evaluate the consistency of flow data between existing flow metering devices. Continuity testing is based on developing a flow balance when there are flow meters installed on all inflow and outflow streams. The sum of the measured inflows is compared to the sum of the measured outflows.

Totalized or average flow values recorded over a minimum of 30 minutes are generally used during the continuity test. Recorded instantaneous flow values, especially in open channels, can vary significantly from one instant to the next due to turbulence, waves, or surges, and are therefore not used for continuity testing. The recorded total inflow of a control volume is compared to the total outflow. In general, the flow metering devices are considered to be consistent when the difference between the recorded inflow and outflow is less than approximately 15 percent. Generally, continuity testing is performed at various flow rates and, if possible, with different combinations of flow meters, to ensure that consistency is maintained over a range of flows.

While continuity testing can provide an evaluation of the flow data between flow meters, it cannot be used to assess the accuracy of an individual flow meter's reading. For example, continuity testing may indicate that a STP's influent and effluent flow meters are recording consistent flow data; however, both meters may be under or over-representing the actual flow rate by the same margin of error.

Therefore, the accuracy of individual flow meters should be evaluated regularly, and any adjustments made through calibration to bring the readings to within the required level of accuracy. Calibration techniques are discussed in Section 9.1.4.

9.1.3 Common Installation Problems and Impacts on Flow Data

The Instrumentation Testing Association (ITA) has published a designer checklist for flow meters in STP applications (ITA, 1999) as well as the proceedings of workshops on the appropriate selection, installation and calibration of flow meters (ITA, 2002).

Open Channel Flow Meters

Issues with the fabrication or construction of the primary flow metering device, such as errors with weir lengths, notched weir angles, flume dimensions, etc., can result in erroneous flow measurements if standard mathematical relationships are used to describe the head versus flow rate relationship. In addition, the incorrect installation of a primary flow metering device, such as non-level weir or flume insert, may also introduce errors. In such cases, onsite calibration would be required to determine the head versus flow rate relationship specific to the particular primary device.

Weirs and flumes require free flowing conditions to make accurate measurements. If the nappe on the downstream side of a weir is not properly aerated, or if free-fall conditions do not exist downstream of a weir, inaccurate flow measurements will be recorded. Similarly, if free-flowing conditions do not exist downstream of a flume, flooding of the weir throat will result in inaccurate flow measurements. In some instances, weirs and flumes can be used to measure flows at flooded conditions; however, two level sensors would be required and a flooded weir or flume equation would need to be used.

The improper location of secondary devices used to measure liquid level can also result in flow data error. For example, if placed too close to a weir, the recorded level would be within the drawdown zone of flow, and would result in a recorded flow which is less than the actual flow. The proper installation location of secondary devices varies with each installation, and should follow the recommendation of the manufacturer and be determined on a case-by-case basis.

Turbulence, waves, and surges in the approach channel can result in erratic head measurements and inaccurate flow data. Structural modifications could be made to reduce turbulence and surging in the approach channel, however this is not always possible. A stilling well may be used to reduce the impact on secondary (level) measuring devices. Stilling wells may require heating to avoid freezing during the winter. Stilling wells cannot be used for area-velocity flow meters which have a combined Doppler / level sensor, as the Doppler sensor needs to be submerged in the flow.

Incorrect zero flow settings can also lead to erroneous data. Depending on the zero error, flows will be over- or under-estimated over the entire flow range. Most primary metering weirs and flumes have non-linear flow versus head curves, and zero errors result in increased errors at higher flows. An assessment of the zero setpoint should be conducted as part of a field calibration (Section 9.1.4).

In addition to following the required calibration schedule, the installation of a staff gauge on the side of the channel at the location of the secondary (level measurement) device, and zeroed to primary device zero, would allow for visual level readings that can be compared to the value determined by the secondary device.

Figure 9-2 - Example Installation of a Parshall Flume with a Staff Gauge at Secondary Device Location

Finally, improper maintenance of weirs, flumes, and secondary devices can contribute to erroneous flow measurements. The accumulation of debris, upstream, downstream, on weirs, or in flumes can result in additional head losses which would lead to erroneously high flow measurements. The condition of the weir plate itself, including increased roughness of the upstream face or dulling of the sharp weir crest, can also result in flow measurement errors. All sensors, whether submerged or installed above the liquid surface, should be frequently inspected to ensure no debris has accumulated which might interfere with measurements.

More information regarding open channel flow meters can be found in Grant and Dawson (1997).

Closed Conduit Flow Meters

One of the most common installation problems that can impact flow measurement is the presence of distortions in the flow profile through the flow meter. Straight runs are required upstream and downstream of closed conduit flow meters so that the flow profile can fully develop prior to the flow meter location. The presence of bends, tees, valves, and other fittings in the closed conduit can distort the flow profile, resulting in readings that are too high, too low, or erratic. The required upstream and downstream straight-run lengths vary from one flow meter to another. Installation should follow manufacturer's instructions and recommendations. Manufacturers should be contacted to determine the recommended installation requirements if special or unusual situations are encountered.

For magmeters, erroneous flow measurements may be encountered when the velocity of flow through the meter is outside the magmeter's operating range. Proper selection of flow meter diameter, based on expected operating flows, is necessary to ensure accurate flow readings.

For differential pressure type flow meters, such as the Venturi meter, erroneous flow measurements may be encountered when the primary device is operating outside of its operating range, although the secondary device (pressure transducer) is operating within its operating range. For example, the pressure transducer associated with a Venturi meter may be capable of accurately measuring low pressure differentials. However, the Venturi meter may not be capable of producing pressure differentials that are a function of the rate of flow at low flow rates. As a result, the operating range of both the primary and secondary devices should be taken into account when collecting flow data.

Finally, deposits accumulating inside closed conduits upstream, downstream and within the flow meters, can also affect accuracy. Where access is possible, this can be diagnosed by visually inspecting the inside of the closed conduit.

9.1.4 Field Calibration Methods

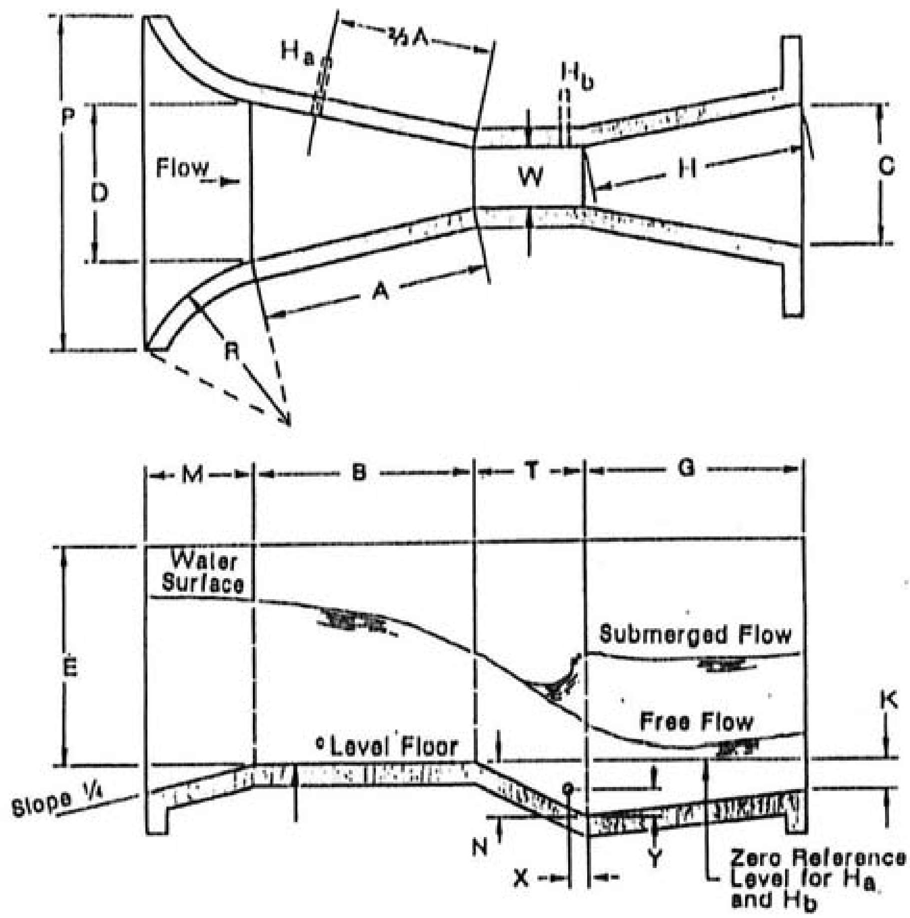
Field calibration, as the name implies, involves calibrating a flow meter in its installed location. Generally, a physical inspection is performed as a first step of the field calibration process, to ensure that the installation is appropriate and to identify any factors that may affect the meter's accuracy. After the physical inspection, one or more calibration techniques can be used to confirm the accuracy of the meter.

Physical Inspection

The purpose of the physical inspection is to note any installation or condition issues that may impact the accuracy of the flow meter of interest or the ability to calibrate it. Typical inspection activities include:

- For weirs and flumes:
  - Physical measurements including the approach channel, weir / flume, and effluent channel dimensions and elevations;
  - Condition of the primary device, including any debris accumulation or weir deterioration; and
  - Location and condition of the secondary measuring device.
- For full conduit flow meters:
  - Meter orientation; and
  - Configuration of piping, valving, and other appurtenances, upstream and downstream of the meter which may contribute to distortion of the velocity profile through the meter.

Figure 9-3 presents example results for the physical measurements of a Parshall flume installation.



Comparison of Field Measurements versus Standard Flume Dimensions				
Parameter*	2-foot Flume		6-foot Flume	
	Standard Dimension**	Field Measurement	Standard Dimension**	Field Measurement
W (inlet)	24	24	72	71.62
W (outlet)	24	24	72	71.25
A	60	60	84	120
2/3A	40	40	54	51.75
B	58.9	58	82.4	76
C	36	36	84	84
D	47.5	48	105	103
E	36	48	36	48
T	24	24.5	24	24
G	36	42	36	36

All values in inches
** From "Isco Open Channel Flow Measurement Handbook".

Figure 9-3 - Physical Inspection of a Parshall Flume Installation

(From MOEE *et al.*, 1996)

Calibration – Dye Testing Techniques

The dye dilution technique utilizes a tracer, such as Rhodamine WT, which is added upstream of the flow meter at a known and constant flowrate. Samples are collected downstream of the flow meter and analyzed for dye concentration. Flow meter readings are recorded when each sample is taken. The dye concentration measured is proportional to the flow rate through the flow meter.

Dye should be added in a zone upstream of the flow meter that provides good mixing, so that the concentration in the sample collected downstream of the flow meter is a representative dilution, and can provide meaningful results.

The dye dilution technique requires complete mixing between the injection point and the sample collection point. The accuracy of this technique is also affected by the dye preparation and injection procedures, as accurate dye feed rates and concentrations are required to determine the actual flow rate.

Calibration – Draw and Fill Technique

This is the simplest method for evaluating flow meter accuracy. The draw and fill technique involves emptying or filling a tank or basin of known dimensions while recording the liquid level in the tank or basin and the flow rate recorded by the flow meter.

Depending on the configuration of the system, measurements can be done either during the draw-down phase (if the flow meter is on the discharge side of the tank or basin), or during the fill phase (if the flow meter is on the influent side of the tank or basin). If possible, all other influent and effluent flows to the test basin should be halted during the collection of measurements.

In some instances, such as a gravity sewer inflow into a pumping station wet well, these additional flows cannot be halted. In such a case, the gravity sewer flow rates may be considered to be fairly consistent during and after the draw-down phase, and can thus be estimated based on the measured increase in liquid level after the draw-down phase. This information can then be used to account for the impact of the gravity sewer flow on data collected during the draw-down phase.

If possible, this technique should be repeated at several flow rates, ideally at the low-, mid-, and high-points of the potential flow meter operating range to check accuracy over the entire range.

Calibration – Redundant Instrumentation

In some instances, it may be possible, by closing valves and/or gates, to configure a system such that the same flow rate is measured by two or more flow meters in series. This is similar in concept to the continuity testing described in Section 9.1.2; however, in this case, a flow meter which is known to be accurate can be used to calibrate a suspect flow meter. Any time delay between changes in flow between the two flow meters needs to be taken into consideration.

Portable flow metering devices such as temporary velocity-area flow meters for open channel flow or ‘strap-on’ Doppler meters for closed conduit flow, can be used to calibrate a permanently installed flow meter. The accuracy of this technique relies on the accuracy of the temporary installation, and should be used with caution and only if another calibration method is not available.

9.2 OVERVIEW OF SAMPLERS AND SAMPLING METHODS

9.2.1 Purpose of Flow Sampling and Types of Samplers

The purpose of sampling is to accurately characterize the composition of a particular process stream. At sewage treatment facilities, sampling is used to characterize various streams in both the liquid treatment and sludge handling process trains.

Accurate sampling data is essential when undertaking a process evaluation or optimization activities at a STP. Accurate sampling data will provide insight into the actual operating conditions within unit processes, and form a basis with which to evaluate process performance.

Samples can be collected as “grabs”, which provides information regarding the characteristics of the sampled stream at a given instant in time, or as “composites”, which provides information regarding the average characteristics of the sampled stream over a specified period of time. Composite samples are generally a series of grab samples collected at specified intervals which are then mixed together to form a composite sample. Composite samples are sometimes flow corrected, in which the volume of each contributing grab sample is proportional to the flow rate of the sampled stream at the time of sample collection. These types of samples are called “flow-proportional composite samples”.

Samples can be collected manually, or by the use of automatic samplers (autosamplers). In some instances, the installation of an autosampler may not be feasible due to piping and/or channel configurations, or due to intermittent flow of the process stream. In these circumstances, composite samples can be made by combining a series of manually collected grab samples.

Figure 9-4 - A Typical Autosampler

9.2.2 Evaluating Sampling Data

Historic data and trends can be used to evaluate sampling data. Individual sample results can be evaluated for known relationships, such as the value of parameter ratios (such as Total Kjeldahl Nitrogen to 5-day biochemical oxygen demand ($TKN:BOD_5$), TP:TSS, etc.) or the experience that lower than average raw sewage concentrations are observed during wet weather periods. Such an evaluation may indicate if a given sample can be considered to be an outlier, and/or if particular analytical results appear to be suspect.

Sampling data can also be evaluated for its accurate representation of the sampled stream based on typical literature values, operational experience at the facility, and/or mass balances. It should be noted that mass balances, such as a solids mass balance around a clarifier, and per capita loading calculations also rely on flow data (Section 9.1). Therefore, the accuracy of both the sampling data and flow measurements will impact mass balance and per capita loading results.

Typical literature values for domestic raw sewage characteristics and per capita loadings, presented in Table 9-2, could be used to evaluate raw sewage sampling data for a system with little to no industrial users on the collection system. Industrial loadings in the collection system can significantly impact raw sewage characteristics, and, if present, should be taken into consideration when evaluating sampling data.

Table 9-2 - Typical Domestic Raw Sewage Characteristics

Parameter	**Concentration (mg/L)**		**Per Capita Loadings (g/cap/d)**	
	MOE (2008)	**Metcalf & Eddy (2003) (1)**	**MOE (2008)**	**Metcalf & Eddy (2003) (2)**
BOD_5	150 – 200	110 (low) 190 (med) 350 (high)	75	80
TSS	150 – 200	120 (low) 210 (med) 400 (high)	90	90
TP	6 – 8	4 (low) 7 (med) 12 (high)	3.2	3.2
TKN	30 – 40	20 (low) 40 (med) 70 (high)	15.6	13
TAN	20 – 25	–	10.1	7.6

Notes:

1. The “low”, “med”, and “high” refer to low, medium, and high strength sewages. Low strength wastewaters based on an approximate per capita flowrate of 750 L/cap/d, medium strength on 460 L/cap/d, and high strength on 240 L/cap/d.
2. Typical values without ground-up kitchen waste.

9.2.3 Common Sampling Problems and Impacts on Sample Data

Poor selection of a sampling location can result in the collection of samples that do not accurately represent the stream being sampled. Stagnant areas, or those subject to eddying or backflow, should be avoided.

The composition of various process streams can vary diurnally; therefore, sampling bias can be introduced if samples are consistently collected at the same time of day. This bias can be avoided by collecting 24-hour composite samples for continuously flowing streams, or ensuring that grab samples are collected at varying times of the day.

For intermittently flowing streams, such as primary sludge or waste activated sludge (WAS), sample bias may be introduced if grab samples are consistently collected at a specific point in the discharge period (such as at the beginning or end of a pumping cycle). A sample collected in this way will represent the composition of the stream at that point in the discharge period, rather than the average composition. The impact of this bias can be mitigated by collecting a series of grab samples throughout the discharge period, which are then combined to form a composite sample.

In some instances, intermittently flowing streams, such as septage, digester supernatant, and filter backwash waste, are discharged into the treatment process upstream of a sampling location. Depending on the time that samples are collected, the impact of these streams may not be included, or may be over represented, in the collected sample. If possible, the frequency and timing of the discharge of these streams should be assessed against the sample collection schedule.

The handling and storage of samples can also negatively impact sample data. Samples should be properly stored and analyzed in a timely manner to ensure accurate analytical results. Information regarding preservation methods and sample holding times can be found in APHA/AWWA/WEF (2005).

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CHAPTER 10

PRELIMINARY TREATMENT

CHAPTER 10

PRELIMINARY TREATMENT

10.1 SCREENING AND GRINDING

10.1.1 Types of Screening and Grinding Equipment and Common Problems

The purpose of screening is to remove coarse materials from raw sewage to prevent damage to downstream mechanical equipment, and minimize the accumulation of coarse solids such as rags in downstream channels and tanks.

Screens are typically among the first unit processes encountered in a sewage treatment plant. A screen is a device with openings through which raw sewage flows. Coarse screens provide protection for downstream unit processes and equipment from blockage and physical damage (MOE, 2008).

There are three general categories of screens based on the size of the openings. Coarse screens have openings ranging in size from 6 to 25 mm. The opening sizes in fine screens range from 1 to 6 mm and microscreens have openings less than 1 mm. Coarse and fine screens are typically used in preliminary treatment whereas microscreens are typically used for effluent screening (Metcalf & Eddy, 2003). This section focuses on the optimization of coarse and fine screens used for preliminary treatment.

Coarse screens are often constructed of parallel bars or rods, called a bar rack, and are used for the removal of coarse solids. Fine screens are typically comprised of perforated plates, wires, or mesh. The solids captured by the screens are called screenings, and may be removed manually or via a mechanical cleaning mechanism. Screens are subject to issues relating to screenings drainage, bagging and odour control.

In lieu of coarse screening, grinding or comminution may be utilized. Grinders, comminutors and macerators serve to cut or chop coarse solids in the raw sewage into smaller particles. The use of grinding is advantageous when used to protect downstream processes from large objects and to eliminate the need to handle screenings. Comminutors have the tendency to produce strings of material such as rags that can negatively impact downstream processes and equipment. As a result, the use of grinders, comminutors and macerators has become less common as plant operators and designers prefer technologies that remove the material from the sewage stream (MOE, 2008).

Table 10-1 presents some of the symptoms and common problems encountered with screening and grinding.

Table 10-1 – Screening and Grinding – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Screen/grinder channel overflow or bypassing.	• Raw sewage bypasses the screens/grinders, potentially transporting coarse material to and damaging downstream unit processes and equipment • Plugging of downstream pumps or equipment with rags	• Screens are blinded (Section 10.1.3) • Screens/grinders or channels inadequately sized • Mechanical or power failure of grinder
Uneven flow distribution to screens/grinders.	• Some screens/grinders are hydraulically overloaded, overflowing, or operating at a headloss greater than the design headloss	• One or more screens are blinded (Section 10.1.3) • Poor hydraulics of upstream flow control devices • Mechanical or power failure of one or more grinder units
Increased screenings quantities.	• Greater than normal quantities of screenings collected	• Fat, oil and grease can accumulate on the screens • Combined sewer systems typically produce larger quantities of screenings than separate sewer systems
Lower screenings quantity than expected.	• Lower than expected quantities of screenings collected	• Oversized screen openings • Raw sewage contains low concentrations of large debris and coarse materials

10.1.2 Evaluating Performance

Table 10-2 presents monitoring recommended, in terms of sampling locations and analyses, in order to evaluate the performance of the screens and grinders.

Table 10-2 – Screening and Grinding – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Upstream and Downstream of Screens/Grinders	Level Measurement	• Headloss across screens/grinders	The maximum operating headloss across a screen unit is usually identified by the equipment supplier.
Screenings Bin	Quantity Measurement	• Mass of screenings • Volume of screenings	The quantity of screenings depends on the screen type, and type of collection system.

Figure 10-1 presents a process schematic of typical screen and grinder arrangements.

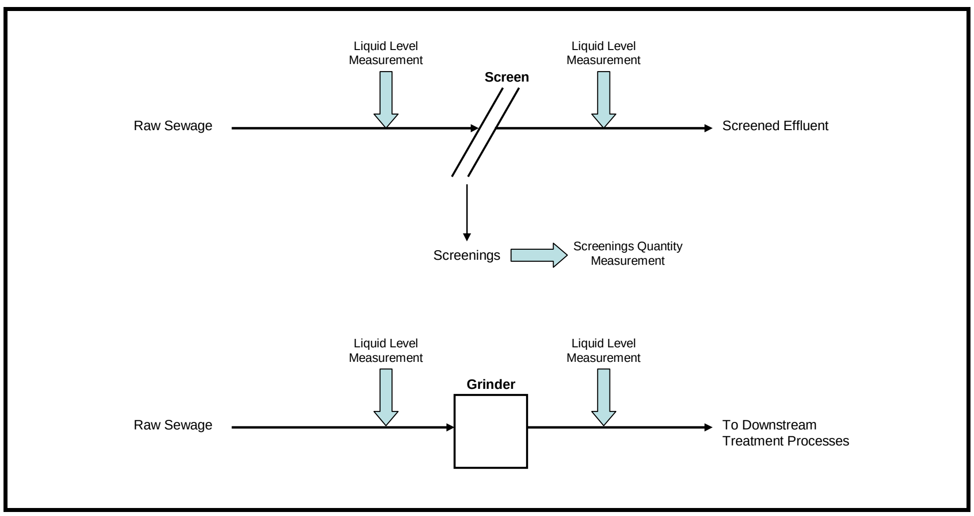

Figure 10-1 - Screening and Grinding – Process Schematic and Recommended Sampling and Monitoring Locations

Typically, screen performance is evaluated based on the achieved removal of screenings. Because grinders do not remove any solids from the sewage stream for treatment, operating performance is not easily measured quantitatively.

Table 10-3 presents typical process performance for various screen types and sizes.

Table 10-3 - Screening - Typical Process Performance for Screens Treating Raw Sewage

(Adapted from Metcalf & Eddy, 2003)

Screen Type (1)	Screenings Moisture Content (%)	Screenings Specific Weight (kg/m^3)	Volume of Screenings per Volume of Sewage Treated (L/ 1,000 m^3)	
			Range	Typical
Coarse Bar Screen, 6 – 25 mm Openings	60 – 90	700 – 1,100	37 – 74	50
Fine Bar Screen, 1 – 6 mm Openings	80 – 90	900 – 1,100	44 – 110	75
Rotary Drum, 6.5 mm Openings	80 – 90	900 – 1,100	30 – 60	45

Notes:

1. For typical performance of other types of screens used in sewage treatment, equipment suppliers should be contacted.

10.1.3 Optimization Techniques

Screen Cleaning

As a screen traps coarse material and debris, the screen develops more resistance to the flow of sewage through the openings. This increases water levels upstream of the screen and the overall headloss experienced across the screen. The increased water levels upstream of the screen can lead to:

- bypass if the screen channel is equipped with a bypass; or
- overflow of the screen channel or upstream of the screens.

In the case of bypass, coarse materials may be carried to downstream processes potentially damaging equipment and negatively affecting operation and performance. Channel overflows may result in additional action and reporting by operations staff. Where multiple screens are utilized, blinding of one or more of the screens may result in uneven flow to the screens and lead to uneven wear of screening and cleaning equipment.

Therefore, the purpose of optimizing screen cleaning is to prevent bypassing or overflow, while maintaining process performance.

Screen cleaning may be achieved by manual cleaning or with automated cleaning mechanisms. Manual cleaning is regularly performed by an operator from an accessible platform with suitable drainage. The major deficiencies with manual cleaning occur when there is no operator on site (i.e. overnight and weekends), or in the event that the cleaning platform becomes inaccessible (i.e. flooding). In both cases, the screens may remain blinded for extended durations, adversely affecting downstream processes and equipment. The installation of automatic mechanical cleaning mechanisms may serve to reduce the occurrence of extended periods of blinding. Typically, the automated systems are operated on a schedule designed to maintain the headloss across the screens within a range identified by the manufacturer. Alternatively, the cleaning mechanisms may also be set to automatically engage when the headloss reaches a specific upper limit, ensuring that the headloss across the screens remains below a maximum value set by the manufacturer (FCM and NRC, 2003).

10.2 GRIT REMOVAL

10.2.1 Types of Grit Removal Equipment and Common Problems

Grit removal facilities are typically located downstream of screens and upstream of grinders and primary clarifiers. Grit removal is provided to protect downstream unit processes and equipment from damage and grit accumulation.

Grit removal in Ontario is typically accomplished via grit channels, aerated grit tanks or vortex units (MOE, 2008). Grit channels are typically employed in smaller plants and consist of an unaerated channel. The grit channel is typically designed such that the length and depth of the channel promote the settling and removal of grit particles. In small plants, grit channels are usually manually cleaned.

Aerated grit tanks utilize air to induce spiral flow in the sewage perpendicular to the flow through the tank. The heavier particles settle while the generally lighter, organic particles are carried through the tank (WEF/ASCE, 1998).

Vortex grit removal units work by inducing vortex flow patterns in the sewage. Heavier grit particles settle by gravity in the bottom of the unit while organic particles exit with the effluent (Metcalf & Eddy, 2003). Efficient operation depends on ensuring proper velocity and flow split as determined by the manufacturer or supplier. Where grit removal is employed, grit drainage, removal, and odour control are all potential issues of concern that should be considered.

Table 10-4 presents the symptoms and common problems encountered with grit removal.

Table 10-4 – Grit Removal – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Uneven flow distribution to grit removal units.	• Uneven grit collection • Frequent and consistent clogging of particular units/pumps	• Poor hydraulics of upstream flow control devices
Grit accumulation in downstream channels and tanks.	• Increased frequency of maintenance of downstream pumps as a result of increased wear due to grit • Frequent unit process shut down for removal of accumulated grit in downstream tanks and channels	• Grit removal equipment inadequately sized • Insufficient aeration and/or ineffective aeration pattern in aerated grit tanks (Section 10.2.3) • Sewage velocity is too great in grit channels or aerated grit tanks to allow settling of grit particles (Section 10.2.3)

10.2.2 Evaluating Performance

Table 10-5 presents monitoring recommended, in terms of sampling location and analyses, in order to evaluate the performance of the grit removal system.

Table 10-5 – Grit Removal – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Grit Storage	Quantity Measurement	• Volume of grit	The quantity of grit depends on the type of collection system.

Figure 10-2 presents a process schematic of a typical grit removal arrangement.

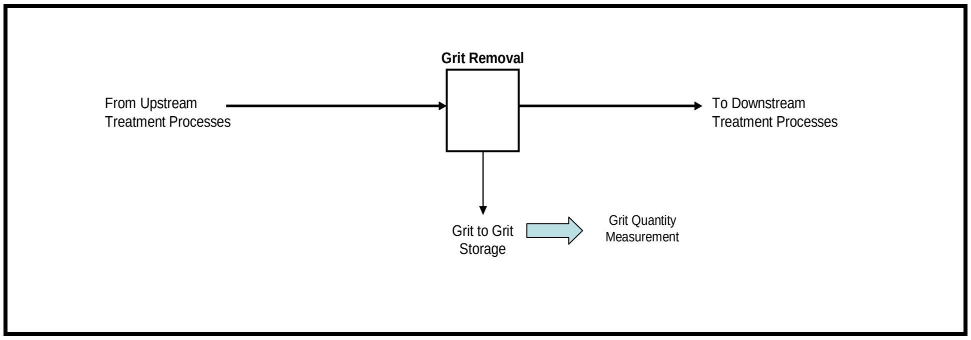

Figure 10-2 – Grit Removal – Process Schematic and Recommended Sampling Locations

Typically, grit removal performance is evaluated based on the achieved removal of grit.

Table 10-6 presents typical process performance for combined and separate sewer systems.

Table 10-6 – Grit Removal – Typical Performance

(Adapted from MOE, 2008 and Metcalf & Eddy, 2003)

Collection System	Typical Grit Quantity per Volume of Sewage Treated (m^3/ 1,000 m^3)
Combined Sewers	0.004 – 0.18
Separate Sewers	0.004 – 0.037

10.2.3 Optimization Techniques

Aeration in Aerated Grit Tanks

The roll induced by the aeration determines the size of particles removed in an aerated grit tank. If the intensity of aeration is too great, grit will be carried out of the tank in the effluent. If the intensity of aeration is too low, organic material will be removed in addition to grit, affecting downstream processes and resulting in odours in the collected grit.

Particles of differing settling velocities can be selectively removed by adjusting the aeration rate to the aerated grit tanks. Increasing the aeration will increase the minimum settling velocity of the grit particles that will be removed. Increasing the minimum settling velocity will effectively reduce the amount of grit removed. Similarly, decreasing the aeration will decrease the minimum particle settling velocity of the grit particle that will be removed in the grit tank. In other words, decreasing the minimum settling velocity will increase the amount of grit removed.

Introducing tapered aeration can promote even and more efficient grit removal along the length of aerated grit tanks. This may be accomplished by providing separate valves and flow meters for individual banks of diffusers, and/or dedicated blowers for grit tank aeration for added control.

Optimizing Hydraulics of Grit Removal Units

Installation of influent and/or effluent baffles is frequently used to control hydraulics through the grit tank and to improve grit removal effectiveness. Baffling reduces the potential for short-circuiting in grit tanks (FCM and NRC, 2003).

Sewage velocities through grit channels may be controlled by adjustments to the effluent weirs. Where hydraulically possible, changes to the effluent weir height or type can effectively modify the sewage velocity through grit tanks.

The installation or adjustment of rotating paddles or flow control baffles in vortex units can maintain quasi-constant flow velocity during low flow periods. This can effectively maintain the grit removal efficiency over a wider range of flows.

In plants with multiple grit removal units, adjusting the number of units in operation by controlling the flow through gates can serve to ensure that the grit removal units are operating at peak efficiency at all times.

10.3 CASE HISTORIES

10.3.1 Greater Augusta Utility District WWTP, The Greater Augusta Utility District, Maine – Affordable Modifications

The following case study is based on the information presented in Burbano *et al*. (2009).

Background and Objectives

The Greater Augusta Utility District WWTP is a conventional activated sludge plant consisting of screening, grit removal, primary and secondary treatment, and disinfection prior to discharge to the Kennebec River. The WWTP has an average daily flow (ADF) rated capacity of 15,000 m^3/d (4 mgd).

Grit removal consists of two aerated grit chambers, each with a volume of approximately 170 m^3. The bottom of each chamber has a 25 percent slope towards the screw collector mechanism. Sewage enters the aerated grit chambers and travels along the length of the chamber in a spiral pattern induced by the coarse bubble diffusers. Baffling at the end of the chamber is designed to produce a quiescent zone to promote settling and prevent grit from being carried over the effluent weir. The grit chambers are operated in parallel, with air blowers at low speed during typical flows and shut off during high flow events. Figure

10-3 presents the existing configuration of an aerated grit chamber at the Greater Augusta Utility District WWTP.

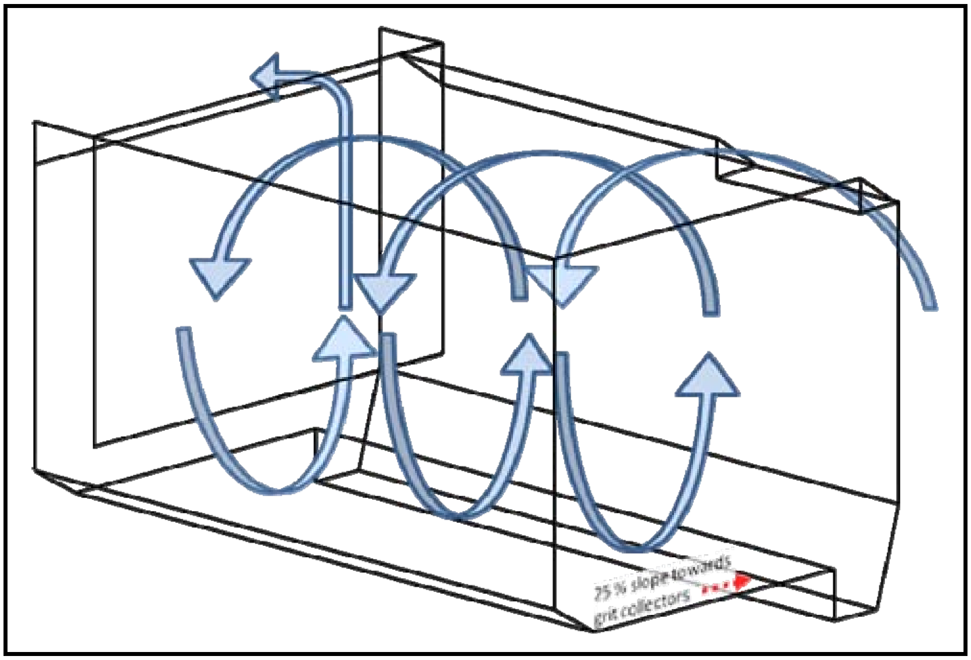

Figure 10-3 - Existing Aerated Grit Chamber Configuration

During high flow events (flows in excess of 38,000 m^3/d to 45,000 m^3/d), significant grit carryover has been observed. During storm events, flows can reach 151,000 m^3/d, exacerbating the problem in conjunction with the increased grit loading due to the combined sewage flow. Conversely, at low flows (11,400 m^3/d), organics tend to settle in the grit chambers.

The significant grit carryover has resulted in significant operational and maintenance issues. As a result of the poor grit removal performance, there has been considerable grit deposition in downstream unit process tankage, clogged primary sludge piping and excessive wear on primary sludge pumps.

As part of a long-term CSO program, The Greater Augusta Utility District identified deficiencies in the grit removal process at the WWTP for improvement. It was determined that replacement of the aerated grit chambers was financially infeasible. As a result, the objective of this study was to develop in-tank modification options to optimize the aerated grit chamber performance.

Optimization Methodology and Results

CFD modelling was utilized for the analysis of potential modifications to the aerated grit chambers. Several models were developed based on updated design guidelines, and modifications to baffling and other structural elements of the grit chambers. Mass and momentum flux, discrete particle trajectory, and turbulence models and equations were incorporated to model the flow through the chamber. The discrete-phase model was used to simulate the effect of aeration on the sewage flow pattern. The effect on grit removal was not included in the model, but was evaluated based on the velocity and flow-patterns output by the CFD model.

The model was initially run with the existing chamber geometry and flow to simulate historic conditions and to observe the flow patterns resulting in the observed grit carryover. Modifications to the baffling configuration and chamber geometry were then modelled to observe the resultant changes in flow patterns. The following summarize the results of the CFD modelling.

Configuration 1: Existing Aerated Grit Chamber Configuration

Non-ideal flow conditions are predominant during high flow events. The existing baffling forces flow below the baffle and then vertically towards the effluent weir. The effect of the baffling brings the flow into close proximity to the grit collection system near the effluent end of the chamber, resuspending grit and then carrying the solids over the effluent weir.

Configuration 2: Removal of Existing Baffling

Removal of the existing baffle eliminates the vertical flow at the end of the chamber. This change in flow pattern allows the desired spiral flow pattern to continue along the length of chamber. As a result, the model predicts a reduction in grit carryover.

Configuration 3: Two Lateral Baffles

This proposed modification would remove the existing baffling, and install two baffles laterally one-third and two-thirds along the length of the chamber. These baffles would not extend from wall-to-wall or from floor-to-surface. Rather, the baffles would be located centrally across the cross section of the chamber. These baffles are designed to affect flow along the center of the induced spiral flow pattern, preventing short-circuiting along the center of the spiral. The results of the CFD modelling indicated that although the diameter of the center of the spiral increased, no significant changes in hydraulics were identified. It is expected that grit contact with the new baffles would result in an increase in grit removal.

Configuration 4: Longitudinal Baffling

The installation of vertical, longitudinal baffling serves the purpose of isolating the area of upward flow in the spiral pattern due to aeration to minimize resuspension of grit, and to reduce the effective chamber width. The CFD modelling indicated that at intermediate flows, the longitudinal baffling served to reduce the upward velocity of the spiral flow through the chamber. As a result, decreased grit carryover is expected at intermediate flow rates. At high flows, there was no significant improvement in flow pattern over Configuration 2.

Configuration 5: Removal of Existing Baffling and Increased Aerated Grit Chamber Floor Slope

The floor of the existing aerated grit chambers was designed with a 25 percent slope towards the grit collection mechanism. Recent changes included in the latest version of *Design of Municipal Wastewater Treatment Plants* (WEF/ASCE, 2009) recommend a minimum slope of 30 percent in order to facilitate grit removal and to minimize grit carryover. The results of the CFD modelling with a 30 percent slope indicate that there were no significant observable improvements in flow pattern through the aerated grit chamber.

Results

Based on the results of the CFD modelling and analysis, Configurations 2 and 4 were selected for implementation. The existing lateral baffle was removed and a new, fibreglass enforced, plastic longitudinal baffle was installed in each aerated grit chamber. At the time of reporting, plant operations staff have observed marked improvements in grit removal system performance.

10.3.2 Renton Treatment Plant, Municipality of Metropolitan Seattle, Washington – Optimization of Grit Removal at a WWTP

The following case study is based on the information presented in Finger and Parrick (1980).

The Renton Treatment Plant is a conventional activated sludge plant consisting of screening, grit removal, primary and secondary treatment, and disinfection. The plant has a rated capacity of 138,000 m^3/d.

Grit removal at the Renton Treatment Plant is achieved in two pre-aeration tanks. Grit collected in the pre-aeration tanks is pumped to four cyclone grit separators for separation prior to dewatering and ultimate disposal.

Over time, ineffective grit removal had resulted in the accumulation of grit in downstream process channels, tanks and digesters. This resulted in increased pump wear, frequent maintenance and process interruptions, increased loading and significant costs for removal of grit from the channels.

Several specific problem areas with grit removal were identified, including:

- The cyclone grit separators were prone to clogging at increased flows, occurring two or three times over the course of an eight hour shift. The frequent clogging required operations staff to shut down and disassemble the clogged unit to manually remove the clogged material;

- Uneven loading to the grit sumps resulted in excessive loading to select grit pumps, and clogging due to heavy grit accumulation and the intake of plastics and sticks (also responsible for the clogging of the cyclones). The uneven loading resulted in increased pump runtimes for the affected grit pumps;

- Grit accumulated in the primary distribution channel, eventually interfering with the operation of the channel aeration. Eventually, the grit began to slough into the primary clarifiers. In order to resume operation of the swingfusers, the channel and four associated primary clarifiers had to be taken offline for several days. The channel had to be drained and cleaned manually, costing approximately 70 labour-hours; and

- Grit accumulation in the primary clarifiers resulted in the premature wear of the progressive cavity sludge pumps. Inefficient grit removal resulted in reconditioning costs of $2,500 USD per unit for parts.

An optimization program was undertaken to correct operational problems linked to deficiencies in the grit removal system.

The installation of a grit screen on the overflow of two of the four grit cyclones virtually eliminated the issue of cyclone clogging. By eliminating the rags and other coarse material from the system the bar screen effectively reduced the frequent clogging of the grit cyclones and pumps.

Tapered aeration in the grit tanks was introduced via a combination of sparger replacement and increased air flow control. An extreme taper was introduced, resulting in significantly improved grit removal.

The tapered aeration also resulted in more even distribution of grit loading along the length of the pre-aeration tank, and more even grit pump runtimes. In conjunction with the installation of the bar screens at the grit cyclones, the issue of grit pump clogging was virtually eliminated.

The increased grit removal efficiency translated into a significant reduction in grit accumulation in the primary distribution channel and primary clarifiers. Additionally, a reduction of 30 to 40 percent in the amount of wear on the raw sludge pumps was estimated as a result of the reduction of grit in the raw sludge.

Based on the minor modifications and operational changes, the plant managed to reduce operational problems and increase grit removal efficiency while avoiding potentially costly upgrades.

10.4 REFERENCES

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CHAPTER 11

PRIMARY CLARIFICATION

CHAPTER 11

PRIMARY CLARIFICATION

11.1 OVERVIEW OF PRIMARY CLARIFICATION

11.1.1 Purpose of Primary Clarification and Types of Primary Clarifiers

The purpose of primary clarification is to remove readily settleable solids and floating material, such as grease and scum from sewage. In addition to removing solids, particulate BOD_5 is also removed. As a result, the primary clarification process reduces the organic and solids loading on downstream treatment processes.

Primary clarifiers are either circular or rectangular tanks. Baffling is normally installed to promote solids settling by providing quiescent conditions within the clarifier. Sludge collection mechanisms are used to remove the raw sludge that accumulates on the bottom of the tank, while skimmer mechanisms are used to remove scum that accumulates on the liquid surface. Information regarding the design of primary clarifiers can be found in Chapter 11 of the *Design Guidelines for Sewage Works* (MOE, 2008).

Chemicals, such as coagulants and/or polymers, can be added upstream of primary clarifiers to enhance solids and/or phosphorus removal. This process is referred to as Chemically Enhanced Primary Treatment (CEPT, Section 11.2). In addition, WAS from the secondary treatment system can be added upstream of the primary clarifiers, in which case the WAS is “co-thickened” with the raw sludge.

This chapter focuses on optimizing primary clarifier performance in terms of improving sedimentation performance. Optimization of the primary clarification process can involve modifying flow control structures (such as effluent weirs and baffles within the clarifiers) and operational practices (such as raw sludge pumping frequency or chemical dosage) to improve the performance of the primary clarifiers with respect to solids removal.

11.1.2 Evaluating Process Performance

Table 11-1 presents a typical primary clarifier monitoring program, in terms of sampling locations and analyses that would be used to evaluate the performance of the process.

Table 11-1– Primary Clarification – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Primary Influent	Composite Recommended	• Flow rate • BOD_5 • TSS • TP (suggested) • TKN (suggested)	In the absence of significant impacts due to internal recycle streams such as digester supernatant, raw sewage samples can normally be assumed to be equivalent to primary clarifier influent.
Primary Effluent	Composite Recommended	• BOD_5 • TSS • TP (suggested) • TKN (suggested)	Primary effluent samples should be collected on the same days as primary influent samples so that removal efficiencies across the primary clarifiers can be calculated.
Sludge Blanket	Discrete	• Sludge blanket depth	Commonly accomplished using a “Sludge Judge”, hand-held solids analyzer, or on-line sludge blanket monitor. It is recommended that sludge blanket readings be taken at various longitudinal locations (for rectangular clarifiers) or radial locations (for circular clarifiers) to develop a sludge blanket profile.
Settled Sludge	Composite Recommended	• Volume • Total solids (TS) • Total volatile solids (TVS)	Composite samples can be collected as a series of grab samples throughout the duration of a sludge pumping cycle. It is recommended that samples be collected at different times during the day so that results are not biased towards operational conditions specific to certain times of day.

In addition to the recommended sample locations and analyses presented in Figure 11-1, it is recommended that the following also be monitored:

- Quantity and quality of WAS discharged upstream of primary clarifier(s), if co-thickening is practiced;
- Quantity and quality of other process waste streams, such as digester supernatant, added upstream of the primary clarifier(s), if applicable;
- Quantity and quality of hauled wastes, such as septage, added upstream of the primary clarifier(s), if applicable;
- Quantity and characteristics of coagulants and/or polymers added upstream of the primary clarifier(s), if applicable; and
- Raw wastewater and primary clarifier effluent soluble phosphorus concentrations if coagulants are being added upstream of the primary clarifiers for phosphorus removal.

Figure 11-1 presents a process schematic of a typical primary clarification process, along with the identification of various sampling locations.

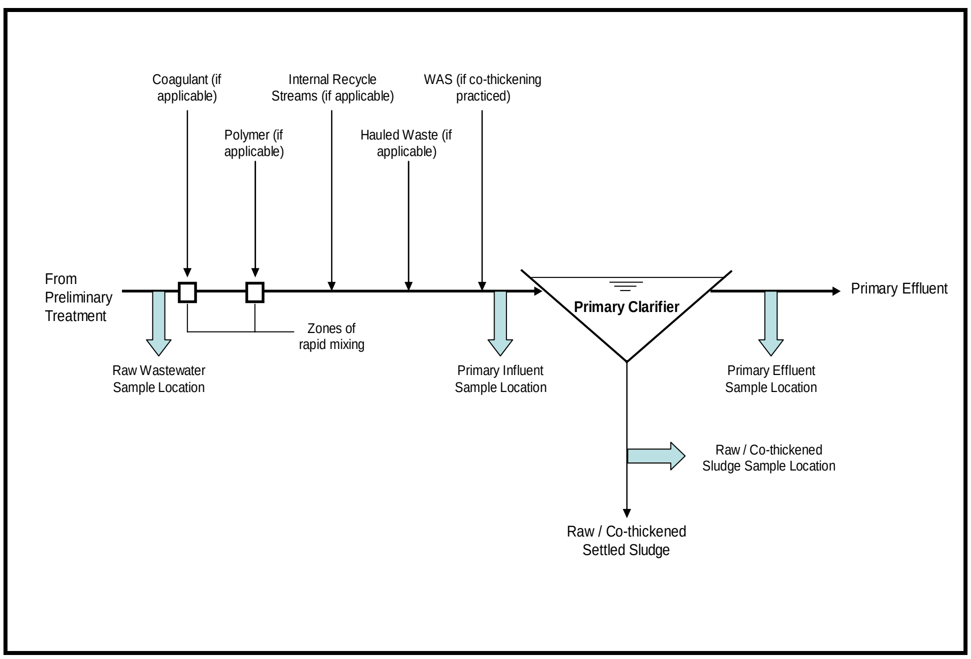

Figure 11-1 – Primary Clarification – Process Schematic and Sampling Locations

Typically, primary clarifier performance is evaluated based on the achieved removal efficiencies of BOD_5 and TSS, and the concentration of sludge withdrawn from the clarifiers. Table 11-2 presents typical process performance for the primary clarifiers for various operating conditions.

Table 11-2 – Primary Clarification – Typical Process Performance

Operating Condition	Typical BOD_5 Removal (%)	Typical TSS Removal (%)	Typical Raw Sludge Solids Concentration (%)
Without upstream chemical addition for phosphorus removal.	25 – 40 [1, 2]	40 – 70 [1, 2]	n/a
With upstream chemical addition for phosphorus removal.	45 – 80 [1, 2]	60 – 90 [1, 2]	n/a
Without WAS co-thickening.	n/a	n/a	4 – 12 [2]
With WAS co-thickening.	n/a	n/a	2 – 6 [2]

Notes:

n/a – not applicable

1. MOE (2008).
2. Metcalf & Eddy (2003).

It should be noted that the BOD_5 and TSS removal efficiencies presented in Table 11-2 have generally been calculated based on the recorded concentrations of these constituents in the raw sewage and primary effluent. In many instances, raw sewage characteristics can be assumed to be representative of primary influent characteristics (WERF, 2006). However, caution should be exercised when attempting to apply typical process performance values when internal recycle streams, WAS, and/or chemical addition result in significant variations between raw sewage and primary influent characteristics (WEF/ASCE, 1998). In such cases, it may be beneficial to determine the BOD_5 and TSS removal efficiencies based on concentrations in the primary influent and primary effluent. In some instances, due to the location of addition points for internal recycle streams, polymers, and/or coagulants, it may not be possible to collect primary influent samples that include contributions from these streams. In such cases, raw sewage sample data, and the quantity and quality of the recycle streams, polymers, and/or coagulants should be recorded so that the composition of the primary influent can be estimated based on a mass balance approach.

11.1.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the primary clarification process are shown in Table 11-3.

Table 11-3 – Primary Clarification – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Uneven flow distribution between clarifiers.	• Some clarifiers are overloaded, potentially resulting in poor effluent quality due to limited settling • Other clarifiers are underloaded, potentially resulting in stagnant, septic conditions, reducing effluent quality due to rising septic sludge and/or causing odours • Uneven rate of effluent flow between primary clarifiers visible in effluent launders	• Uneven weir levels between clarifiers • Uneven weir lengths between clarifiers • Poor hydraulics of upstream flow control devices
Hydraulic short-circuiting within clarifiers.	• Reduced primary clarifier effluent quality • Stagnant, septic regions and regions of high flow and poor settling within clarifier • Erratic clarifier performance	• Poor design of inlet structures and in-clarifier baffling (Section 11.4.2) • Density currents due to temperature gradients, and wind-driven circulation cells (Section 11.4.1)
Long sludge retention time.	• Development of septic, rising sludge, reducing primary effluent quality and potentially causing odours • Deep sludge blanket, resulting in decreased effluent quality due to solids carryover, especially during high flow events	• Poor control of raw sludge pumping – insufficient pumping (Section 11.3)
Short sludge retention time.	• Low raw sludge TS concentrations, resulting in increased hydraulic loading on solids handling processes • Little to no sludge blanket	• Poor control of raw sludge pumping – excessive pumping (Section 11.3)

Table 11-3 – Primary Clarification – Symptoms and Causes of Common Problems (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Poor clarifier performance not attributable to problems identified above.	• Removal efficiencies below typical removal rates, resulting in poor effluent quality	• Characteristics of primary influent not conducive to good settling performance (Section 11.5.2) • Clarifiers hydraulically overloaded from operating at flows exceeding their design values • Clarifiers hydraulically underloaded from operating at flows significantly below their design values • WAS pumping practices resulting in high instantaneous solids loadings on clarifiers practicing co-thickening (Chapter 20) • Scum carry-over due to poor performance of scum collection system and/or improper scum baffle installation • Cold sewage temperatures leading to reduced settling rates

11.2 CHEMICALLY ENHANCED PRIMARY TREATMENT (CEPT)

11.2.1 Purpose and Impact on Performance

CEPT utilizes chemical coagulation and flocculation to enhance the performance of the primary clarifiers by increasing the fraction of settleable solids and improving the settleability of these solids. CEPT is generally used to enhance performance of the primary clarification process in two ways, namely:

- To increase removal efficiencies of TSS, BOD_5, and, depending on the coagulant used, possibly TP; and
- To allow existing clarifiers to be operated at higher surface overflow rates (SORs) while maintaining effluent quality.

The chemicals used in CEPT may be categorized as coagulants and flocculants. It has been found in some studies that the addition of a flocculant alone did not enhance primary clarifier performance (Chack *et al*., 1994; Neupane *et al*., 2006).

Therefore, CEPT is best achieved either through the addition of a coagulant alone, or a coagulant plus a flocculant. The types and roles of coagulants and flocculants are described below.

- Coagulants: These are generally aluminum or iron metal salts. The purpose of coagulant addition is to form insoluble metal hydroxide precipitants. As the metal hydroxides settle, a sweeping effect is produced, where colloidal particles in the sewage are entrapped by the metal hydroxide floc and are removed from the sewage. Coagulants also neutralize surface charges on particles, enhancing the flocculation process.

- Flocculants: These are commonly natural and synthetic polymers which are normally used in conjunction with a metal salt coagulant. Polymers may have either a negative charge (anionic), a positive charge (cationic), or an almost neutral charge (non-ionic). The purpose of polymer addition is to form bridges between floc particles, resulting in a larger, stronger, and more readily settleable floc.

It has been reported that CEPT, involving a combination of coagulant and polymer addition, can achieve average TSS removal rates ranging from 60 to 80 percent, and average BOD_5 removal rates ranging from 50 to 57 percent (Gerges *et al*., 2006; Newbigging and Stephenson, 2003; Harleman and Murcott, 1991). In addition, operating SOR values as high as 114 $m^3/m^2/d$ have been achieved without a decrease in TSS and BOD_5 removal efficiencies (Harleman and Murcott, 1991; Mills *et al*., 2006).

It should be noted, however, that the potential impact of implementing CEPT on primary clarifier performance varies from one treatment system to another, due to variations in the composition of primary influent and the configuration of individual treatment facilities. Therefore, it is recommended that jar testing and full-scale trials be conducted to evaluate the potential impact on site-specific process performance.

11.2.2 Implementing CEPT

Typical Chemical Dosages

Due to variations in sewage composition and strength, optimal chemical dosages required for site-specific applications should be determined based on the results of jar testing, and refined based on full-scale operating data. In some instances, it has been found to be useful to vary the coagulant dosage to compensate for diurnal variations in influent quality (Chack *et al*., 1994; Gerges *et al*., 2006). Flow proportioning (flow pacing) of the chemical feed rate can also improve performance while reducing chemical usage and cost.

Reported optimal average iron salt and polymer dosages utilized for full-scale CEPT applications are presented in Table 11-4.

Table 11-4 – CEPT – Reported Optimal Chemical Dosages

Chemical	Reported Optimal Average Dosage
Ferric Salts	3.5 – 14.0 mg/L as Fe [1, 2, 3, 4, 5]
Ferrous Salts	5.0 mg/L as Fe [3]
Polymer	< 1.0 mg/L [1, 2, 3, 4, 5]

Notes:

1. Harleman and Murcott (1991).
2. Chack *et al.* (1994).
3. Neupane *et al.* (2006).
4. Gerges *et al.* (2006).
5. Mills *et al.* (2006).

Mixing and Flocculation Requirements

Chemicals should be subject to initial rapid mixing for uniform dispersion in the sewage stream. Where turbulence is insufficient to provide in-channel mixing, consideration should be given to the installation of tanks and mixers. Based on the results of bench scale testing, the intensity of mixing did not have an impact on the resulting settleable TSS fraction with G values ranging between 100 to 770 s^{-1} (Neupane *et al*., 2006). At very high G values (approximately 10,000 s^{-1}) floc breakup was observed, reducing the settleable TSS fraction (Neupane *et al*., 2006).

Post-coagulant addition flocculation time is required for optimal CEPT performance. Flocculation can occur in both unaerated and aerated channels, as well as grit tanks, pipes, and dedicated flocculation tanks. Sudden turbulence in zones utilized for flocculation should be avoided, if possible, to prevent the breakup of flocs.

Optimal flocculation times for metal salt and polymers are reported to range between 10 to 20 minutes (Neupane *et al*., 2006; Gerges *et al*., 2006; Mills *et al*., 2006). In some cases, it was determined that optimal flocculation was achieved if flocculation time was provided after coagulant addition, but prior to polymer addition; however, addition of polymer directly after flash mixing of coagulant was also successfully used (Parker *et al*., 2004).

Optimal mixing and flocculation times required for site-specific applications should be determined based on the results of jar testing, and refined based on full-scale operating data.

Other Considerations

The addition of metal salts can increase sludge generation through the formation of additional solids in the form of precipitants and the increased removal of colloidal particles. While the addition of polymers will not result in the formation of additional solids, they can impact raw sludge generation rates through increased solids removal efficiencies. In addition, the chemicals used as part of a CEPT process can impact the density of the sludge blanket in the primary clarifiers (Chack *et al*., 1994).

An increase in sludge generation due to the implementation of CEPT can negatively impact downstream sludge handling processes. The capacities of sludge handling processes should be assessed prior to implementing CEPT. After implementation, sludge blanket levels, sludge solids concentrations and volumes, and the performance of sludge handling processes should be monitored and assessed.

While both mixing and flocculation are important components of the CEPT process, it has been found that the intensity and duration of rapid mixing did not have as significant impact on reducing the non-settleable solids fraction as did flocculation time (Neupane *et al*., 2006). Therefore, special care should be taken to ensure that adequate flocculation time is provided, and that turbulent conditions that could result in floc breakup are avoided to fully optimize the performance of a CEPT process. This can be accomplished by conducting full-scale trials utilizing various chemical addition points to identify the optimal locations (Chack *et al*., 1994).

CEPT will result in increased phosphorus removal in the primary clarifiers if alum or iron salts are used. Care must be exercised in selection of the operating coagulant dosage as high doses may result in excessive phosphorus removal and nutrient deficiency in downstream biological treatment process(es). Primary influent and effluent phosphorus concentrations should be monitored to ensure that nutrient deficiency does not occur.

Since CEPT may result in increased BOD_5 removal across the primary clarifiers and decreased BOD_5 loading to the downstream biological treatment process(es), primary effluent BOD_5 concentrations should be monitored to allow for process operational adjustments. Lower BOD_5 loadings to downstream biological treatment processes may result in a decrease in aeration system energy requirements.

The addition of acidic metal salts can reduce the alkalinity in sewage, which may necessitate addition of lime, sodium hydroxide or some other source of alkalinity to maintain the pH. In plants where nitrification is an objective or requirement, the consumption of alkalinity due to the addition of metal salts must be considered in determining the alkalinity available for nitrification.

11.3 SLUDGE PUMPING

11.3.1 Impact on Performance

The purpose of sludge pumping in a primary clarifier is to remove the settled solids that have accumulated at the bottom of the tank. Should these solids not be removed in a timely manner, long sludge retention times within the clarifier could potentially cause several negative process impacts including:

- Development of septic conditions within the sludge blanket, resulting in rising septic sludge, increased odours, and a deterioration of primary effluent quality; and

- High sludge blanket levels, resulting in the potential carryover of solids, especially during high flow conditions, leading to deterioration in primary effluent quality.

If excessive volumes of sludge are pumped out of the primary clarifiers, this can result in decreased raw sludge concentrations and increased hydraulic loadings on downstream sludge handling processes.

Optimizing sludge pumping will prevent a deterioration of primary clarifier or sludge handling system performance. The advantages of optimized sludge pumping include:

- Development of a healthy, non-septic sludge blanket, that allows for adequate thickening of raw sludge prior to pumping;

- Increased process robustness in terms of more consistent effluent quality at both low and high flow conditions; and

- Appropriate hydraulic loading to downstream sludge handling processes.

It should be noted that the optimization of sludge pumping is an ongoing process at a STP. Various factors can influence the optimal sludge pumping requirements for a specific facility, including:

- Changes in raw sewage flows and loadings, due to growth in the service area or seasonal sewage variations;

- Changes in the quality or quantity of internal recycle streams added upstream of the primary clarifier(s), such as sludge processing waste streams and filter backwash flows;

- Changes in the quality or quantity of hauled waste streams added upstream of the primary clarifer(s); and

- Implementation of CEPT, or changes in upstream coagulant and/or polymer dose.

As such, operational data should be used on an ongoing basis to modify sludge pumping rates to ensure continued effective performance of the primary clarifier(s).

11.3.2 Sludge Pumping Optimization Measures

In general, it is desired to maintain a sludge blanket that is sufficiently shallow to avoid the development of septic conditions, while providing adequate thickening of sludge prior to pumping. The most appropriate sludge blanket depth will vary from facility to facility as a result of variances in the characteristics of the solids being settled and the configuration of the clarifiers themselves. In addition, optimal blanket depths can vary from season to season. Therefore, operational experience should be used to determine site-specific optimal sludge blanket target depths.

Optimizing sludge pumping from primary clarifiers involves determining the optimal sludge pumping frequency and rate required to maintain the sludge blanket within target operating depths. Controlling the sludge blanket depth can improve primary clarifier effluent quality, and reduce hydraulic loading on downstream sludge handling processes.

The optimization of sludge pumping is an ongoing process. Sludge blanket depths should be recorded at least daily and compared to the target sludge blanket depth operating range.

In the case that a target depth range has not yet been determined, clarifier performance and raw sludge concentrations should be monitored closely, and the operating sludge blanket depth evaluated for its ability to meet the primary effluent quality requirements, sludge solids concentration, and overall operating targets.

Should the actual sludge blanket depth be within the target range, no change in sludge pumping frequency or rate is required. Should the actual sludge blanket depth be outside the target range, the volume of sludge pumped should be adjusted accordingly. For example, should the sludge blanket be above the target depth, the volume of sludge pumped from the clarifier should be increased, and vice versa. Potential impacts of increased sludge volumes on downstream sludge handling processes should be assessed.

In some instances, difficulty may be encountered when trying to optimize sludge pumping due to the configuration of sludge collection mechanisms within the clarifiers. In these cases, the physical configuration of the sludge collection mechanisms may impede the ability to effectively remove solids from the clarifiers. For example, the “rathole” effect can sometimes be encountered during sludge pumping wherein sewage is drawn into the sludge pump suction line through a “hole” that develops in the collected sludge (Metcalf & Eddy, 2003). This can be diagnosed by collecting a series of grab samples during a pump cycle and evaluating the solids content of each grab sample.

Such a problem may require the reconfiguration or upgrading of the sludge collection mechanism to fully optimize clarifier sludge pumping. Implementation of continuous sludge pumping, as opposed to cycled sludge pumping, may help to alleviate these types of problems. Where the implementation of continuous sludge pumping is not feasible, the pump cycle duration and/or pumping rate can be reduced, while increasing the frequency of pump cycles. Variable diurnal sludge pumping patterns can also be used to optimize sludge removal.

Online sludge density sensors, installed on sludge pump discharge piping, can also be used to control the duration of sludge pumping cycles. With this type of configuration, sludge pumping continues until the concentration of solids in the raw sludge stream is less than a predetermined set-point. This reduces the potential for the accumulation of solids in the clarifier due to insufficient sludge pumping, and reduces the potential for hydraulic overloading of downstream solids handling processes due to excess sludge pumping. The City of London is currently utilizing such a system with great success at the Greenway Pollution Control Centre (Fitzgerald, 2009).

Other means available to optimize sludge pumping include utilizing continuous online sludge blanket detectors to automatically control sludge pumping cycles. The Instrument Testing Association (ITA) has tested sludge blanket detectors for use in STPs. Test results are available through the ITA website www.instrument.org.

If possible, sludge pumping should be conducted on a daily basis. Some facilities operate with sludge pumping during weekdays only, relying on the primary clarifiers to provide solids storage capacity over the weekend. This practice can lead to deterioration in primary effluent quality, especially during periods of high sludge blanket depth, and high hydraulic loadings on sludge handling processes during sludge pumping periods.

In addition to recording sludge blanket depths, it is also recommended that sludge pump cycle frequency, volumes of sludge pumped, and sludge concentrations be recorded on a daily basis. These data can be used to assess the accuracy of recorded measurements by conducting a solids mass balance around the primary clarifier(s). Should a large discrepancy between predicted and recorded sludge wasting rates be found, the data sources, including flow meters, pump runtime meters, and sample collection protocols, can be evaluated for potential sources of error.

11.4 SHORT-CIRCUITING

11.4.1 Causes and Impacts on Performance

In an ideal clarifier, all influent would have the same hydraulic residence time within the clarifier, equal to the ratio of the volume of the clarifier to the influent flow rate. In practice, however, clarifiers are subject to non-ideal flow conditions.

Short-circuiting occurs when a portion of flow reaches the outlet of the clarifier prior to the bulk of the flow that entered the clarifier at the same time. Short-circuiting can lead to deterioration in clarifier performance due to a reduction in effective clarifier volume available for sedimentation, and potential solids carryover due to localized velocity gradients.

Possible causes of short-circuiting within a clarifier include:

- Inefficient clarifier design. The design of the clarifier may itself result in short-circuiting of flow, for example, due to inadequate or misplaced baffling, and uneven effluent weirs;

- Density currents. These form due to temperature differences, and resulting density differences, between the influent and the contents of the clarifier. If the influent flow is more or less dense than the contents of the clarifier, it will tend to flow across the bottom and top of the clarifier respectively. This phenomenon is sometimes referred to as thermal stratification; and

- Wind-driven circulation cells. As the name implies, these are created by wind blowing across the liquid surface, resulting in the formation of circulation cells that significantly reduce the useable volumetric capacity of the clarifier. This is most often observed in shallow clarifiers.

The extent of short-circuiting within a clarifier can be evaluated by conducting tracer testing (Section 11.5.1). The results of multiple tracer tests can be used to identify if the clarifier hydraulics are consistent (similar tracer response curves between tests) or erratic (dissimilar tracer response curves between tests).

If consistently poor clarifier hydraulic performance is observed, then there is likely a design limitation affecting the clarifier's performance. Installation of baffles, or modification of inlet structures, may be able to improve clarifier performance (Section 11.4.2).

If multiple tracer tests identify erratic hydraulics within the clarifier, it is more likely that clarifier performance is being impacted by density currents or wind-driven circulation cells.

11.4.2 Inlet Structures, Outlet Structures and Baffling

Horizontal variations in velocity along the width of the clarifier can lead to flow short-circuiting (WEF/ASCE, 1998). The presence of these velocity gradients is generally propagated at either the inlet structures or outlet structures of the clarifier. The in-tank hydraulic performance can vary with flowrate, resulting in a deterioration of hydraulic performance at high flow rates.

Inlet structures must be designed to provide sufficient velocity to avoid solids deposition in the influent channel, while providing sufficient velocity dissipation to provide quiescent conditions within the tank. This can be achieved through the use of inlet baffles or diffusers.

Figure 11-2 presents various designs of conventional center-feed circular primary clarifiers.

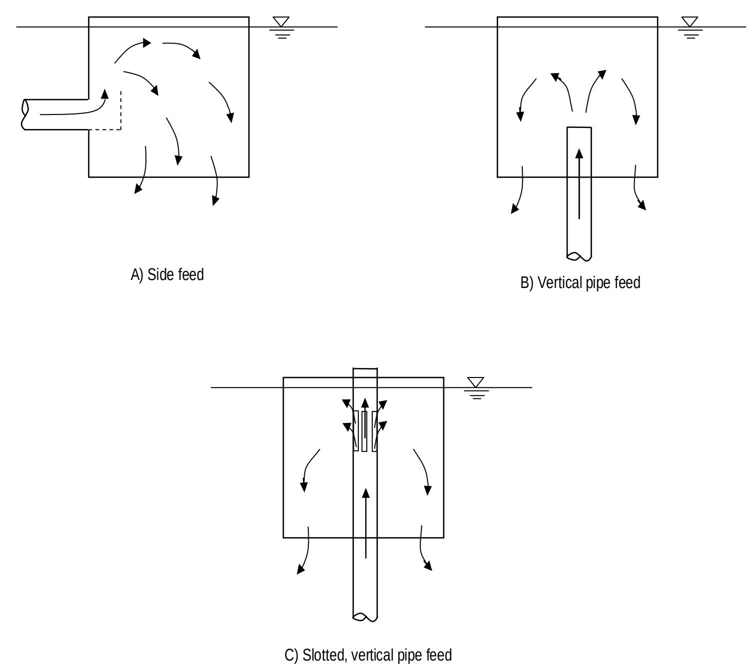

Figure 11-2 – Typical Inlet Structures of Circular Center-Feed Clarifiers

(Adapted from WEF/ASCE, 1998)

In the case of rectangular clarifiers, several inlet ports are generally provided to equalize influent flow along the width of the clarifier. Examples of such configurations can be found in WEF/ASCE (1998). Clarifier performance can be improved by ensuring equal flow distribution through the inlet ports. Flow equalization can be improved by locating inlet ports away from tank walls, and modifying the inlet baffles or diffusers based on the inlet channel hydraulics (WEF/ASCE, 1998).

Outlet structures should be configured such that effluent is withdrawn uniformly to avoid localized velocity gradients. Effluent weirs should be level and firmly attached to the effluent structure to avoid uneven flow along the weir length. Replacement of straight edged weirs with V-notched weirs may improve performance as imperfectly levelled V-notched weirs are not as susceptible to non-uniform flow as imperfectly levelled straight edged weirs (WEF/ASCE, 1998). In addition, in “squircle” clarifiers, there is the potential for the development of localized velocity gradients at the corners of the square effluent troughs. This can sometimes be alleviated by providing weir blanking in those areas.

Hydraulic modelling packages can be used to evaluate existing in-tank hydraulics. Calibrated models can then be used to project the impact of inlet and outlet structure upgrades or modifications on clarifier performance (see Chapter 6). After modifications are implemented, tracer testing can be used to confirm the impact of these changes on hydraulic performance.

11.5 FIELD INVESTIGATIONS

The following sections outline field investigations that can be used to identify process limitations, or to evaluate the impact of implementing optimization measures on process performance.

11.5.1 Tracer Testing

Tracer testing methods available for evaluating clarifier hydraulic performance are outlined in Chapter 14.

11.5.2 Determining Ideal Clarifier Performance

The characteristics of primary influent, including contributions from raw sewage, hauled waste, and internal recycle streams, can be evaluated to determine the ideal performance expected from a primary clarifier.

Since a primary clarifier relies on sedimentation to remove suspended solids and particulate organic matter, only the settleable fraction of these constituents can be removed during the primary clarification process. Because the composition of primary influent, in terms of settleable and non-settleable TSS, BOD_5 and COD fractions, can vary significantly on a diurnal, weekly or seasonal basis (WERF, 2006), ideal clarifier performance will also vary. In addition, differences in primary influent composition from one treatment plant to another will result in different achievable primary clarifier performance for the respective system.

A method has been developed to assess the impact of sewage characteristics on primary clarifier performance (WERF, 2006). In addition to identifying ideal clarifier performance, methods are outlined to quantify the total inefficiency, flocculation inefficiency, and hydraulic inefficiency of test clarifiers.

The protocol outlined in WERF (2006) was based on the collection and testing of primary clarifier influent samples collected downstream of any chemical, internal recycle stream and/or WAS addition points. However, when performing testing to evaluate ideal clarifier performance, it is also recommended to collect samples of raw sewage, upstream of any chemical, internal recycle stream and/or WAS addition points, to evaluate the impact of these streams on the potential performance of the clarifier(s) (Wahlberg, 2009). For information regarding the details of required testing and data analysis, reference should be made to WERF (2006).

11.6 CASE HISTORIES

11.6.1 Cornwall STP, Cornwall, Ontario – Optimization of Primary Clarifier Hydraulics

The following case study is based on information presented in Newbigging and Stephenson (2006).

Background and Objectives

It was desired to optimize the Cornwall STP, a primary treatment facility with an average daily flow capacity of 54,400 m^3/d, to ensure that the plant could handle the peak design flow of 108,864 m^3/d.

Treatment at the Cornwall STP consisted of screening, aerated grit removal, primary clarifiers complete with CEPT utilizing alum and polymer, and chlorine disinfection. Field-testing was conducted to identify optimization opportunities for the preliminary, primary, disinfection, and sludge handling and digestion processes. The following summary focuses on the results of primary clarifier optimization activities. Information regarding optimization activities for other unit processes can be found in Newbigging and Stephenson (2003).

Clarifier Flow Splits

The four rectangular primary clarifiers were each equipped with multiple inlet ports to allow for equal flow distribution across the width of each clarifier. Plant operations staff historically closed three inlet ports to improve flow splits between the clarifiers.

Velocity-area flow meters were installed to measure the flows at each of the open inlet ports to identify any potential for optimization of flow splits between the clarifiers. The results of the testing indicated that, while not ideal, the observed flow splits were optimal based on the layout of the clarifiers.

Testing was also done in the vicinity of one of the normally closed inlet ports of Clarifier No. 3. Results indicated that the current approach of closing inlet ports was beneficial in improving flow splits between the clarifiers.

Clarifier Short-Circuiting

Dye testing was used to determine the extent of short-circuiting within the existing clarifiers.

Dispersion testing was performed on Clarifier Nos. 3 and 4. Samples were collected at the first and third launders in Clarifier No. 3, the third launder in Clarifier No. 4, and the combined effluent from Clarifier Nos. 3 and 4. The first launder was located closest to the inlet of the clarifier, while the third launder was located at the end of the clarifier.

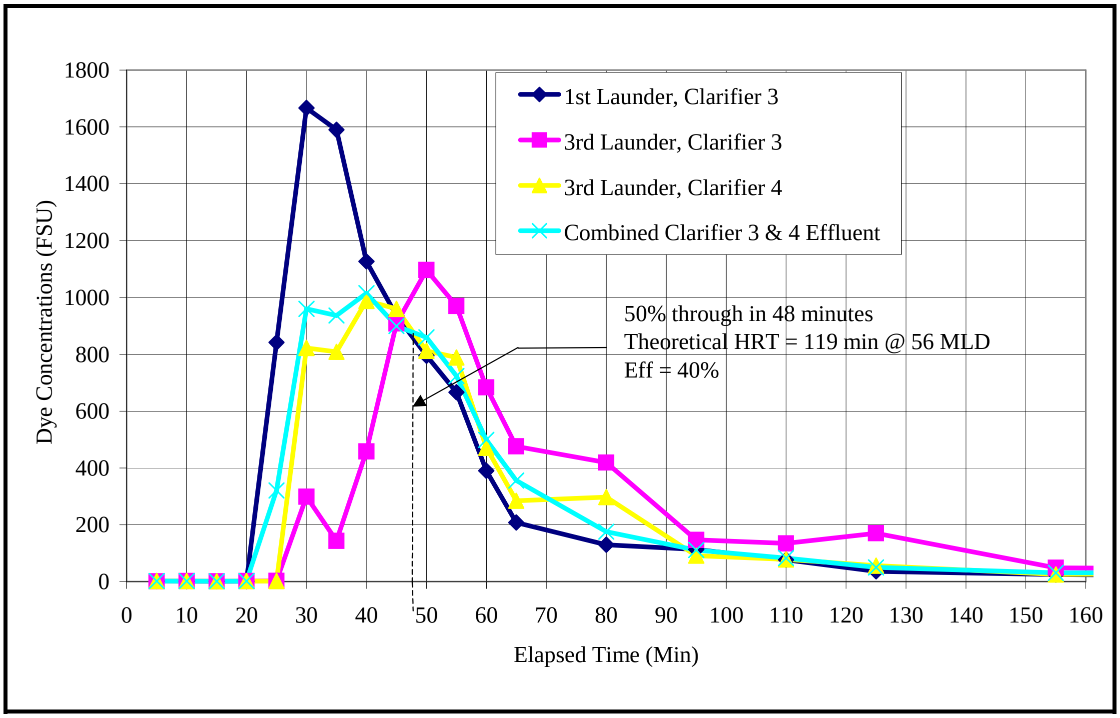

Figure 11-3 – Cornwall WWTP Clarifier Dye Test

(From Newbigging and Stephenson, 2003)

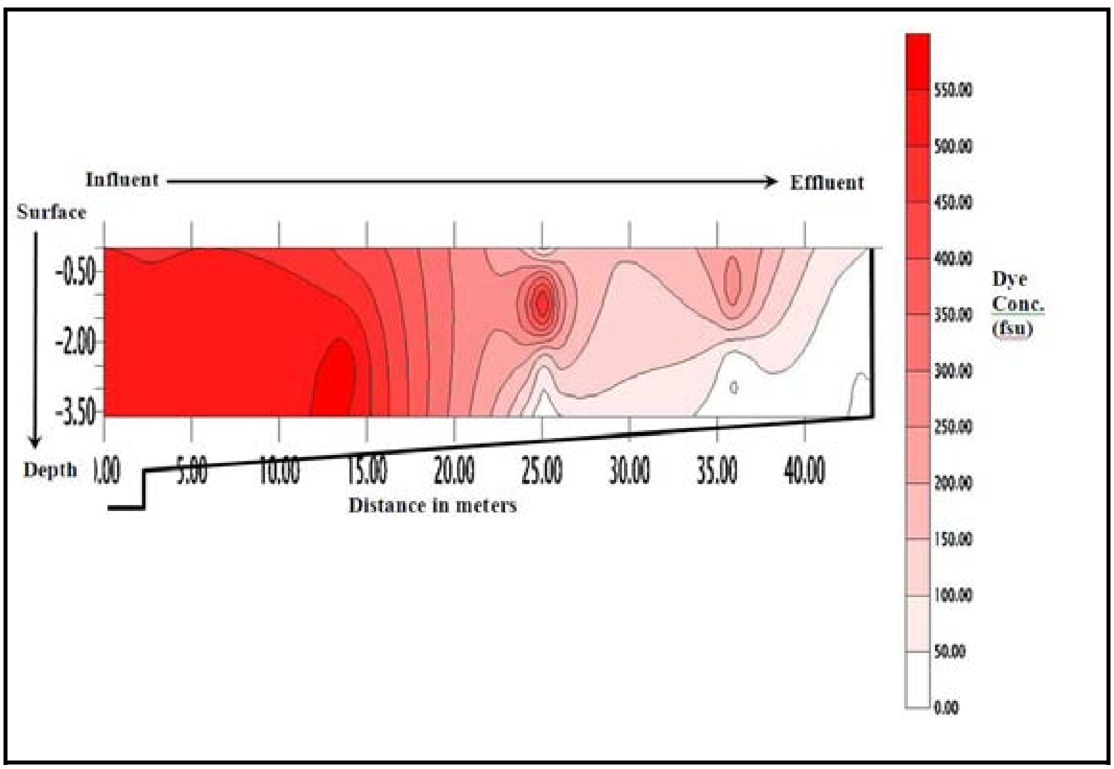

Figure 11-4 – Cornwall WWTP Clarifier Dye Test Flow Pattern Contour Plot

(From Newbigging and Stephenson, 2003)

As shown in Figure 11-3, results of the dispersion testing indicated that dye arrived at the first launder of Clarifier No. 3 well before it arrived at the third launder at the back of the tank. In addition, the dye arrived in Clarifier No. 3 prior to arriving in Clarifier No. 4.

Finally, a flow pattern test was carried out on a primary clarifier. The results (Figure 11-4) confirmed that flow reached the first launder well before the latter launders, resulting in short-circuiting.

The results of the dye testing indicated that additional inlet baffling and mid-length baffling could be used to reduce flow short-circuiting, and improve the overall performance of the clarifiers.

11.6.2 Oro Loma Valley Sanitary District, San Lorenzo, California – Implementation of CEPT

The following case study is based on information presented in Gerges *et al.* (2006).

Background and Objectives

The Oro Loma Valley Sanitary District's STP is a secondary treatment facility providing preliminary, primary, and secondary treatment. Secondary treatment facilities included bioreactors equipped with mechanical aerators.

A project was initiated to restore the capacity of the STP to 75,700 m^3/d (20 mgd) through the implementation of upgrades to the preliminary, primary, and secondary treatment processes.

During pre-design it was identified that, at current primary clarifier removal efficiencies and a design average daily flow of 75,700 m^3/d (20 mgd), the peak wet weather organic loadings to secondary treatment exceeded the capacity of the existing mechanical aerators. Several upgrade options to address this issue were considered, including:

- Upgrading the aeration system in the bioreactors by replacing the mechanical aerators with a fine bubble aeration system or higher capacity mechanical aerators; and

- Increasing removal efficiencies across the primary clarifiers by implementing CEPT to reduce organic loads to the secondary treatment system.

Based on a cost analysis, it was determined that implementing CEPT, if feasible, would be the most cost-effective solution. The objective of this study was to conduct bench-scale and full-scale testing to optimize the implementation of the CEPT process.

Jar Testing

Jar testing was performed to evaluate the potential of CEPT to reduce organic loadings to the secondary treatment stage such that upgrades to the existing aeration system would not be required. Jar testing was based on the addition of ferric chloride and two polymers currently being used at the STP for sludge thickening and dewatering.

Based on the results of jar testing, it was determined that the addition of ferric chloride alone could be capable of sufficiently reducing organic loads, in terms of COD and BOD_5, to the secondary treatment system during wet weather events. Optimal ferric dosages of 10 and 20 mg/L as $FeCl_3$ were determined for low and high primary influent TSS loading conditions, respectively.

Flocculation testing was also undertaken as part of the jar testing program. A 30-second rapid mixing period was followed by a flocculation period ranging from 2.5 to 20 minutes. After the flocculation period, the samples were allowed to settle for 30 minutes and the supernatant was analyzed for TSS, COD, and BOD_5 concentrations. Based on the results of the flocculation testing, the optimal flocculation time was determined to be between 10 and 15 minutes. This information was used to select a chemical dosage point in the plant that would provide sufficient flocculation time during wet weather flow periods.

Finally, settling tests were performed to develop a settling velocity profile. This information was used in conjunction with the High Accuracy Clarifier Model so that TSS removal efficiencies across the primary clarifiers could be projected for different flow rates. Modelling results predicted that the implementation of CEPT would reduce organic loadings to the secondary treatment stage during wet weather events such that upgrades to the existing aeration system would not be required.

Full-Scale Results

Based on the results of the jar testing and process modelling, a full-scale CEPT system was installed in 2005. Intensive field testing then began to confirm the performance of the full-scale system.

It was determined that the results of the jar testing and process modelling were accurate in estimating the effectiveness of the CEPT process and, as a result, no upgrades were necessary to the aeration system in the bioreactors. Based on the difference in estimated costs for the two upgrade options considered, the District saved more than $6.0 million (U.S.) by implementing CEPT.

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CHAPTER 12

BIOLOGICAL TREATMENT

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12.1 OVERVIEW OF BIOLOGICAL TREATMENT

12.1.1 Purpose of Biological Treatment and Types of Bioreactors

If a biological process is properly designed and operated, almost all biodegradable constituents in the sewage can be degraded. The four key objectives of biological sewage treatment as outlined in Metcalf & Eddy (2003) are as follows:

- Oxidize dissolved and particulate biodegradable constituents into acceptable end products;
- Capture and incorporate suspended and non-settleable colloidal solids into biological flocs or biofilms;
- Transform or remove nutrients, such as nitrogen and phosphorus; and
- In some instances, remove trace organic constituents (i.e. microconstituents).

Two distinct types of biological treatment processes have evolved to accomplish the objectives outlined above, namely: suspended growth and fixed film processes. In addition, processes that combine both suspended growth and fixed film attributes have been developed (i.e. hybrid processes).

Continuous Flow Suspended Growth Processes

Suspended growth sewage treatment processes maintain the microorganisms responsible for biological treatment in suspension through mixing. Although mixing can be accomplished mechanically, the most common suspended growth processes involve mixing using aeration. There are a large number of suspended growth processes which have been developed which include, among others, conventional activated sludge, extended aeration, and contact stabilization.

The activated sludge treatment process was developed in 1913 and was given the name because the process produces a mass of microorganisms that are actively able to stabilize sewage under aerobic conditions (Metcalf & Eddy, 2003). Within the aerobic zone of an activated sludge process, biodegradation of the organic constituents in the sewage is accomplished by ensuring adequate mixing to allow the microorganisms or mixed liquor suspended solids (MLSS) to come into contact with both dissolved oxygen and influent sewage. In processes developed to biologically remove nutrients (Section 12.5), anoxic and/or anaerobic zones in combination with aerobic zones can be utilized in a variety of configurations in order to enhance the removal of nitrogen, phosphorus or both.

Following the bioreactor, the mixed liquor flows into a clarifier where settling takes place to separate the active microorganisms from the treated wastewater. A portion of the MLSS is recycled back to the bioreactor to continue to degrade the organic material entering the reactor. The ability of the MLSS to form flocs which can separate and settle within the clarifier is essential to achieving the required level of treatment using an activated sludge process. Depending on the configuration of the systems, suspended growth processes are capable of accomplishing low effluent nitrogen and phosphorus concentrations as is discussed in more detail in Section 12.5.

Further information on suspended growth processes can be found in MOE (2008), WEF/ASCE/EWRI (2005), and Metcalf & Eddy (2003).

Batch Suspended Growth Processes

The Sequencing Batch Reactor (SBR) process is a suspended growth process in which both biological treatment and solid separation take place in one reactor in a series of sequential steps or cycles. The cycles are normally:

- Fill: Substrate is added to the reactor;
- React: Aeration of the influent and biomass for a specific reaction time;
- Settle: Aeration stopped in the reactor to allow clarification through gravity settling;
- Decant: A portion of the clarified effluent is drawn from the reactor; and
- Idle: Idle step required for systems with multiple reactors in operation. In some cases, this step is omitted.

Optimization of SBRs is discussed in Section 12.7.

Fixed Film Processes

Fixed film processes, also known as attached growth processes, utilize microorganisms which are attached or fixed to an inert support material within the bioreactor. Fixed film processes can operate either aerobically, anoxically, or anaerobically depending on the configuration. To achieve effective treatment, the sewage and oxygen (if an aerobic process) must be brought in contact with the attached microorganisms which have formed a layer known as a biofilm on the inert support material contained in the reactor. Depending on the type of attached growth process, the support material can either be completely submerged, partially submerged or have sewage sprayed upon it. Commonly utilized attached growth processes include, among others, trickling filters, rotating biological contactors (RBC), biological aerated filter (BAF) and moving bed biofilm reactors (MBBR).

Optimization of fixed film processes is discussed in Section 12.8.

Hybrid Processes

There are a number of biological processes which combine elements of suspended growth and fixed film processes. Typically the process involves an activated sludge process in which support media is introduced into the bioreactor which is known as an integrated fixed film activated sludge (IFAS) process. There are also hybrid processes which involve separate sequential fixed film and suspended growth processes (i.e. the trickling filter/solids contact process, RBC or MBBR followed by activated sludge).

Benefits of combining the two processes include:

- the ability to increase the biomass without increasing the solids load to the settling process;
- decreasing the reactor's susceptibility to solids washout during wet weather flow events; and
- the ability to reduce bioreactor HRT and maximize treatment capacity.

Further information on IFAS processes can be found in Section 12.8.2.

12.1.2 Evaluating Process Performance

Tables 12-1 to 12-3 present monitoring recommendations, in terms of sampling locations and analyses, to evaluate the performance of the various biological treatment processes. Information specific to sampling and analyses suggested to optimize phosphorus removal can be found in Chapter 16.

Table 12-1 provides recommendations for suspended growth and hybrid processes as they have very similar sampling and analysis requirements. Figure 12-1 illustrates the sampling locations and basic process layout for suspended growth and hybrid processes.

Table 12-2 provides recommendations for fixed film processes. Figure 12-2 presents a schematic illustrating the sampling locations and process layout for fixed film processes.

Table 12-3 provides recommendations for SBR processes. Figure 12-3 presents a process layout for a SBR process illustrating the monitoring locations.

Table 12-1 – Recommended Process Monitoring to Evaluate Suspended Growth / Hybrid Process Performance

Sample Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Bioreactor Influent	Composite Recommended	• Flow rate • BOD_5 • TSS • TKN • TAN • TP • pH	• Sampling recommended at influent to bioreactor • Online monitoring equipment for ortho-phosphorus and nitrogen species can be used for real-time monitoring • Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16
Within Biological Reactor	Representative Grab Sample	• MLSS • MLVSS • pH • Alkalinity • Temperature • DO • Sludge volume index (SVI) • Oxidation-reduction potential (ORP, recommended for BNR processes)	• In biological reactors with more than one zone, sampling should take place in all zones • Online monitoring equipment for ortho-phosphorus and nitrogen species can be used for real-time monitoring • Several DO and temperature readings should be taken at a number of representative locations within the biological reactor/zones; this can involve on-line monitoring
Secondary Effluent	Composite Recommended	• $CBOD_5$ • TSS • TKN • TAN • NO_3-N + NO_2-N • TP • pH • Alkalinity	• Online monitoring equipment for ortho-phosphorus and nitrogen species can be used for real-time monitoring • Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16

In addition to the recommended sample locations and analyses presented in Table 12-1, it is recommended that the following parameters also be monitored:

- WAS flow rates;
- WAS TSS concentrations;
- RAS flow rates;
- RAS TSS concentrations;
- the flow rate of any recycle streams within the process; and
- in hybrid systems, an estimation of the amount of media in the bioreactor as well as a visual inspection of a small amount of the media.

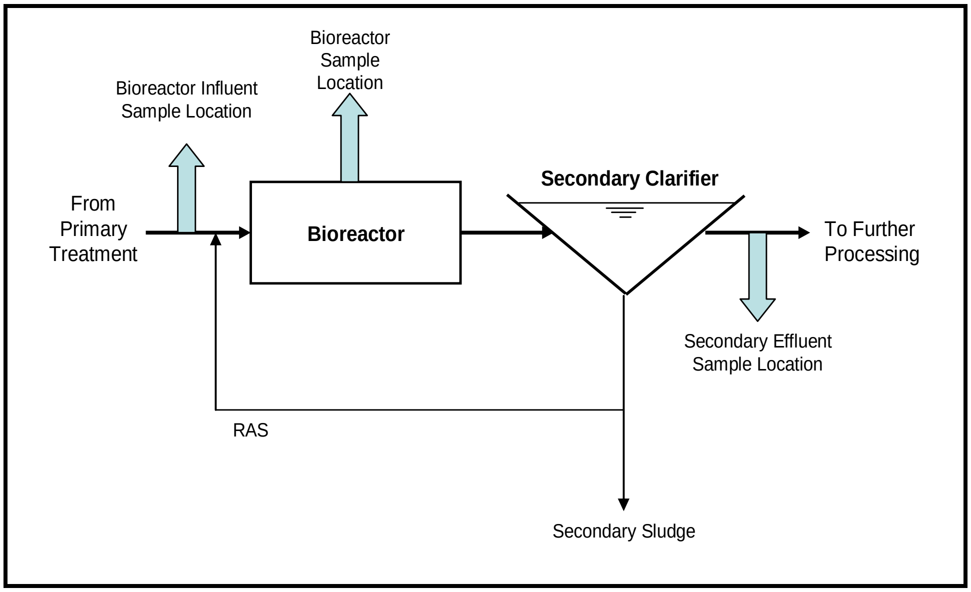

Figure 12-1– Process Schematic and Recommended Sampling Locations for Suspended Growth / Hybrid Processes

Table 12-2 – Recommended Process Monitoring to Evaluate Attached Growth Process Performance

Sample Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Bioreactor Influent	Composite Recommended	• Flow rate • BOD_5 • TSS • TKN • TAN • TP • pH	• Sampling recommended at influent to bioreactor • Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16
Within Biological Reactor	Representative Grab Sample	• pH • Alkalinity • Temperature • DO	• Sampling locations within the stages of an attached growth reactor vary depending on the configuration. For example, in a trickling filter, DO should be measured in the effluent whereas, in RBCs, DO should be measured in the reactor • For non-submerged attached growth systems, sampling may not be possible • In systems with more than one zone or stage, sampling should take place in all zones/stages • Several DO and temperature readings should be taken at a number of representative locations within the biological reactor/zones, where applicable
Secondary Effluent	Composite Recommended	• $CBOD_5$ • TSS • TKN • TAN • NO_3-N+ NO_2-N • TP • pH • Alkalinity	• Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16

In addition to the recommended sample locations and analyses presented in Table 12-2, it is recommended that the following parameters also be monitored when applicable:

- waste sludge flow rates;
- waste sludge TSS;
- the flow rate of any recycle streams within the process; and
- an estimation of the amount of media in the bioreactor as well as a visual inspection of a small amount of the media.

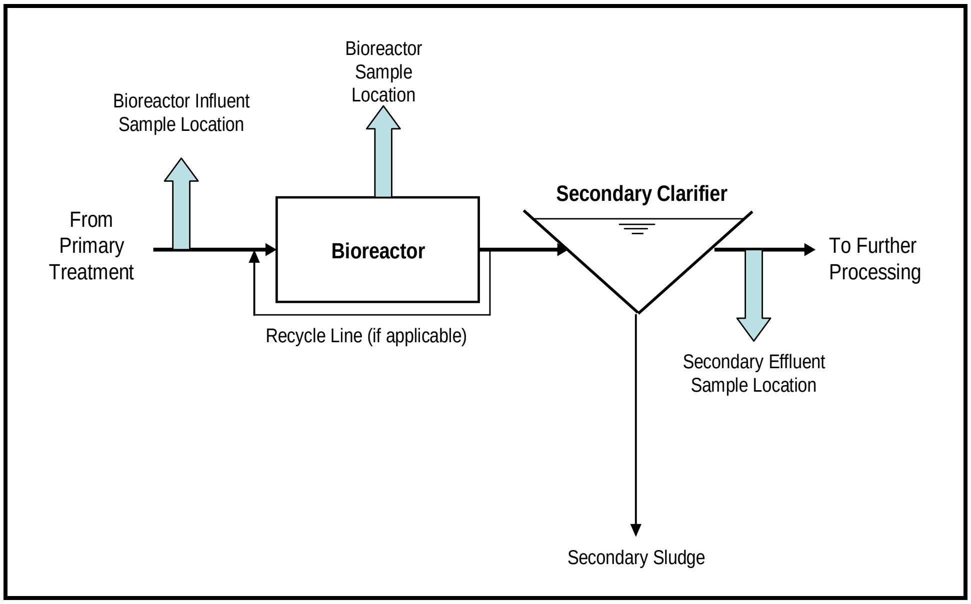

Figure 12-2 – Process Schematic and Recommended Sampling Locations for Attached Growth Processes

Table 12-3 – Recommended Process Monitoring to Evaluate SBR Performance

Sample Timing within Process Sequence	Types of Sample / Measurement	Parameters / Analyses	Comments
Influent	Composite Recommended	• BOD_5 • TSS • TKN • TAN • TP • pH	• Sampling recommended at influent to bioreactor • Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16
During Aeration Mix	Representative Grab Sample	• MLSS • MLVSS • Temperature • SVI • DO • pH • Alkalinity • ORP (suggested for BNR processes)	• Sample should be taken during the mid-point of the aeration cycle to ensure a well-mixed sample is obtained
During Decant Cycle	Grab Sample of Supernatant Above the Settled Sludge	• $CBOD_5$ • TSS • TKN • TAN • NO_3-N+ NO_2-N • pH • TP • ORP (suggested for BNR processes)	• Sample should be taken close to the end of the cycle to ensure that sample is well-settled
Effluent	Composite Recommended	• $CBOD_5$ • TSS • TKN • TAN • NO_3-N + NO_2-N • TP • pH • Alkalinity	• Monitoring in order to improve the phosphorus removal is discussed in detail in Chapter 16

In addition to the recommended sampling locations and analyses presented in Table 12-3, it is recommended that the following parameters also be monitored:

- sludge wasting rate (volume and mass basis) and timing within the cycle;
- influent and decanted volumes per cycle; and
- cycle sequence and duration (i.e. fill time, aeration time, settle time, decant time, idle time).

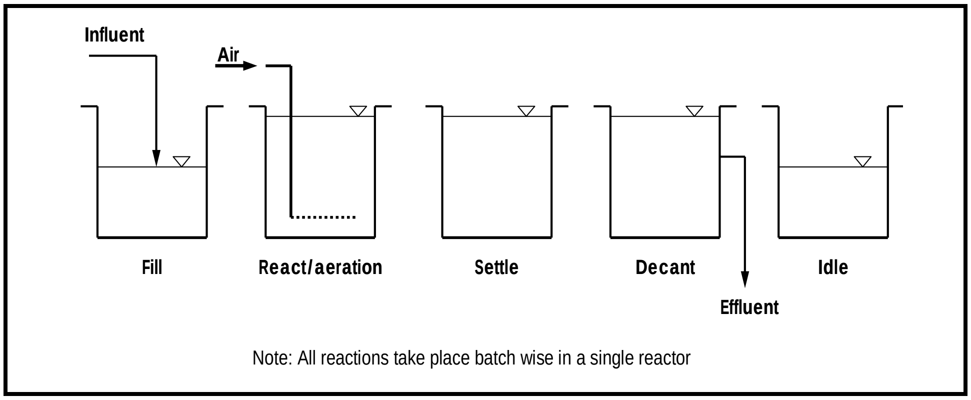

Figure 12-3 – Typical Process Schematic for SBR

12.1.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered within biological treatment processes that can result in lower treatment efficiency are presented in Table 12-4 for suspended growth processes, Table 12-5 for hybrid processes, Table 12-6 for fixed film processes and Table 12-7 for SBR processes.

Table 12-4 – Suspended Growth Processes - Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low BOD removal efficiency.	• Higher than expected $CBOD_5$ effluent concentration	• Low DO (Chapter 13) • Poor bioreactor hydraulics (Section 12.6) • Undersized biological process • Sudden change in influent characteristics • Low MLVSS

Table 12-4 – Suspended Growth Processes - Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Lower than expected nutrient removal efficiency.	• Higher than expected TKN and/or TAN concentration in the effluent	• Low DO (Chapter 13) • Poor nitrification (Section 12.4) • Low HRT within biological process due to short-circuiting (Section 12.6) • Low SRT (Section 12.3) • Low bioreactor temperature which reduces nitrification rate (Section 12.4) • Low alkalinity
	• Higher than expected TN concentration in the effluent	• BOD/TKN ratio too low within zone where denitrification takes place • pH out of optimal range (6-8) within the anoxic zone
	• Higher than expected TP concentration in the effluent	• Poor phosphorus removal (Chapter 16)
Lower than expected solids removal in downstream clarifier(s).	• High SVI • High MLSS within bioreactor • Higher than expected TSS concentration in the secondary effluent	• Sludge foaming and/or bulking issues (Section 12. 2) • Poor SRT Control (Section 12.3) • Poor solids removal efficiency in the secondary clarifier(s) (Chapter 14)
Sudden drop in TAN and/or nitrogen removal.	• Rapid decrease in the nitrogen removal rate in the bioreactor • Recovery of process over a period of time	• Chemical inhibition caused by abrupt change in influent sewage characteristics • Decreased SRT as a result of MLSS loss due to high sludge wasting event or sludge washout
	• Sudden increase in flow rate • Rapid decrease in the nutrient removal rate in the biological treatment process • Recovery of nutrient removal rate over a period of time • Drop in TSS within the zones of the biological treatment processes which coincides with increase in effluent TSS	• MLSS loss due to high wet weather flow (Chapter 7) • Toxic or inhibitory shock load

Table 12-5 – Hybrid Processes - Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low BOD removal efficiency.	• Higher than expected $CBOD_5$ effluent concentration	• Low or uneven DO distribution within reactor (Chapter 13) • Poor bioreactor hydraulics (Section 12.6) • Undersized biological process • Sudden change in influent characteristics, such as due to industrial contributions and/or side stream loadings
Lower than expected nutrient removal efficiency.	• Higher than expected TKN and/or TAN concentration in the effluent	• Inadequate bioreactor mixing preventing sufficient contact between biomass and sewage (Section 12.6 and Chapter 13) • Low or uneven DO distribution within reactor (Chapter 13) • Poor nitrification (Section 12.4) • Low HRT within biological process due to short-circuiting (Section 12.6 and Section 12.8.2) • Low SRT (Section 12.3) • Insufficient media surface area • Low bioreactor temperature which reduces nitrification rate (Section 12.4) • Low alkalinity
	• Higher than expected TN concentration in the effluent	• BOD/TKN ratio too low within zone where denitrification takes place • pH out of optimal range (6-8) within the anoxic zone
	• Higher than expected TP concentration in the effluent	• Poor phosphorus removal (Chapter 16)
Lower than expected solids removal in downstream clarifier(s).	• High SVI • High MLSS within bioreactor • Higher than expected TSS	• Sludge foaming and/or bulking issues (Section 12. 2) • Poor SRT Control (Section 12.3)

Table 12-5 – Hybrid Processes - Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
	concentration in the secondary effluent	• Poor performance of downstream secondary clarifier(s) (Chapter 14)
Sudden drop in TAN and/or nitrogen removal.	• Rapid decrease in the nitrogen removal rate in the bioreactor • Recovery of process over a period of time	• Sudden change in influent characteristics, such as due to industrial contributions and/or side stream loadings
	• Sudden increase in flowrate • Rapid decrease in the TP/ortho-P and nitrogen removal rate in the biological treatment process • Recovery of nutrient removal rate over a period of time • Drop in TSS within the zones of the biological treatment processes which coincides with increase in effluent TSS	• MLSS loss due to high wet weather flow (Chapter 7) • Toxic or inhibitory shock load

Table 12-6 – Fixed Film Processes - Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low BOD removal efficiency.	• Higher than expected $CBOD_5$ effluent concentration	• Low or uneven DO distribution within reactor (Chapter 13) • Poor bioreactor hydraulics (Section 12.6) • Undersized biological process and/or insufficient support media surface area • Sudden change in influent characteristics, such as due to industrial contributions and/or side stream loadings

Table 12-6 – Fixed Film Processes - Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Lower than expected nutrient removal efficiency.	• Higher than expected TKN and/or TAN concentration in the effluent	• Low or uneven DO within fixed film reactor (Chapter 13) • Poor bioreactor mixing preventing sufficient contact of biomass with sewage (Section 12.6 and Chapter 13) • Low HRT due to short - circuiting or uneven flow distribution within fixed film biological process (Section 12.6) • Low air and/or bioreactor temperature which results in reduced nitrification rate (Section 12.4) • Insufficient media surface area • Low alkalinity
	• Higher than expected TN concentration in the effluent	• BOD/TKN ratio too low within zone where denitrification takes place • pH out of optimal range (6-8) within the anoxic zone
	• Higher than expected TP concentration in the clarified/ filtered effluent	• Poor phosphorus uptake within biofilm (Chapter 16) • Poor removal of the sloughed biomass in the downstream solids separation process(es) (Chapter 14 and/or 15)
Lower than expected solids removal.	• Higher than expected TSS concentration in the clarified/ filtered effluent	• Poor solids removal efficiency in the downstream solids separation process(es) (Chapter 14 and/or 15)
Drop in nitrogen removal that occurs suddenly and recovery occurs gradually.	• Rapid decrease in the nitrogen removal rate in the bioreactor • Recovery of process over a period of time	• Chemical inhibition caused by abrupt change in influent sewage characteristics

Table 12-7 – SBR Process - Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low BOD removal efficiency.	• Higher than expected $CBOD_5$ effluent concentration	• Low HRT during the aerobic stage (Section 12.7.1) • Undersized biological process • Sudden change in influent characteristics
Lower than expected nutrient removal efficiency.	• Higher than expected TKN and/or TAN concentration in the effluent	• Low or uneven DO within aerobic stage (Chapter 13) • Poor bioreactor mixing preventing sufficient contact with sewage (Section 12.7.3 and Chapter 13) • Low HRT during the aerobic stage (Section 12.7.1) • Low bioreactor temperature which results in reduced nitrification rate (Section 12.4) • Low alkalinity • Insufficient SRT • Chemical inhibition caused by abrupt change in influent sewage characteristics
	• Higher than expected TN concentration in the effluent	• BOD/TKN ratio too low within anoxic stage where denitrification takes place • pH out of optimal range (6-8) within the anoxic zone • Anoxic stage HRT too low to allow effective denitrification (Section 12.7.1)
	• Higher than expected TP concentration in the effluent	• Poor phosphorus removal (Chapter 16)
Lower than expected solids removal.	• High SVI of the mixed liquor • Higher than expected TSS concentration in the effluent	• Poor SRT control (Section 12.7.2) • Sludge foaming and/or bulking issues (Section 12.2) • Duration of settling cycle not adequate to separate solids (Section 12.7.1) • High rate of effluent decanting causing re-suspension of solids (Section 12.7.1)

12.2 FOAMING AND SLUDGE BULKING

12.2.1 Impact on Process Performance

Suspended growth processes depend not only on the biological oxidation of contaminants but also on the efficient separation of the solids from the liquid, typically by gravity settling. Foaming and bulking of sludge within biological processes negatively impacts the ability of the solids to separate using conventional clarifiers which can then also have a negative impact on the effluent quality from the process. Optimization of secondary clarification is dealt with in Chapter 14. Control of foaming and sludge bulking is discussed in this section.

12.2.2 Common Problems and Causes

Sludge foaming and bulking can result in poor effluent quality and can create problems in downstream solids processes such as thickening and dewatering. Table 12-8 outlines several of the common problems, symptoms and causes of foaming and sludge bulking within suspended growth biological treatment processes. Further information on foaming and sludge bulking can be found in Jenkins *et al.* (2003), and Hossein (2004).

Table 12-8 – Common Problems and Potential Causes of Foaming and Sludge Bulking

(Adapted from Jenkins *et al.*, 2003)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Dispersed microbial growth.	• High turbidity in effluent • High TSS in effluent	• Lack of extracellular polymeric substance (EPS) bridging • High concentration of non-flocculating bacteria • Deflocculation as a result of toxic material and/or poorly biodegradable surfactants
Viscous or non-filamentous sludge bulking.	• Reduced solids separation efficiency • Reduced sludge compaction • Solids overflow from secondary clarifier • Poor sludge dewatering	• High concentration of EPS material associated with zoogloeal growth

Table 12-8 – Common Problems and Potential Causes of Foaming and Sludge Bulking (continued)

(Adapted from Jenkins *et al.*, 2003)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Pin floc.	• Low SVI • Turbid and high TSS effluent	• Breaking and shearing of larger flocs due to turbulence within aeration basin (especially with coarse bubble aeration), pumping, or free fall weirs • Excessive SRT
Filamentous bulking.	• High SVI • Low solids concentrations within the RAS and WAS • Solids overflow from secondary clarifier	• High concentration of filamentous organisms which weakly connect flocs together causing poor compaction, settling and thickening
Rising sludge.	• Scum forms on the surface of the secondary clarifiers and anoxic zones of the aeration basins	• Denitrification takes place releasing nitrogen gas (N_2) which attaches to the biological floc and causes them to float
Foam/scum formation.	• Visible foaming and high TSS on surfaces of treatment tanks	• Undegraded surfactants • High concentrations of Nocardioforms, *M. parvicella* or type 1863 filaments

12.2.3 Microscopic Examinations

Microscopic examination can be used to identify the characteristics of the sludge as well as the microorganisms prevalent within the floc. Examination of sludge under a microscope can be useful in recognizing the physical properties of the flocs and to give insight into how the sludge behaves during solids separation.

The following microscopic techniques can help to identify the cause of a sludge bulking or foaming problem:

- filaments counting;
- floc and filamentous sludge characterization;
- floc and filamentous microorganisms identification; and
- identification of higher level organisms (i.e. protozoa and metazoa).

The information gathered from microscopic evaluations along with sewage characteristics and knowledge of operating conditions can lead to a better understanding of the causes of sludge bulking or foaming.

For more information on the methods utilized in the microscopic examination of sludge, refer to Jenkin *et al.* (2003) and Eikelboom (2000).

12.2.4 Flocculated / Dispersed Suspended Solids Testing

In addition to the evaluation of sludge using microscopic techniques, settling and solids characterization can be helpful in determining the cause, extent and potential solutions to solid separation problems.

There are several tests that can be utilized to quantify the settling characteristics of the sludge. Table 12-9 outlines several of the variations on the sludge volume index (SVI) methodology to quantify the settling characteristics of sludge. SVI gives an indication of the amount of settleable solids within the sludge sample. SVI values of less than 100 ml/g are representative of a well settling sludge and values greater than 150 indicate a poor settling sludge (Metcalf & Eddy, 2003).

In addition, suspended solids testing at the end of the SVI tests can give a greater understanding of the solids characteristics by determining the amount of dispersed suspended solids (DSS) and flocculated suspended solids (FSS). High DSS or FSS concentrations can indicate sludge bulking or settling problems which could require further investigations to determine the cause including the use of microscopy techniques (Section 12.2.3), and a review of the operating conditions.

For more information on the procedures and testing of suspended solids refer to APHA/AWWA/WEF (2005) and Jenkins *et al.*, (2003).

Table 12-9 – Tests Related to Sludge Foaming and Settling
(Adapted from Jenkins *et al.*, 2003; APHA/AWWA/WEF, 2005)

Parameter	Test Procedure	Other Information
Standard Unstirred SVI	• Pour 1 L of freshly sampled mixed liquor into a 1 L graduated cylinder • Allow to settle quiescently out of direct sunlight for 30 min. • After 30 min., record volume occupied by settled sludge • Analyze a separate aliquot of the mixed liquor for TSS	$SVI = \frac{\text{settled sludge volume (ml/L)}}{\text{MLSS (g/L)}}$ • TSS concentration in the supernatant at the end of the test quantifies the DSS

Table 12-9 – Tests Related to Sludge Foaming and Settling (continued)
(Adapted from Jenkins *et al.*, 2003; APHA/AWWA/WEF, 2005)

Parameter	Test Procedure	Other Information
Stirred SVI (SSVI) at a Standard Initial TSS Concentration	• Determine TSS of mixed liquor sampled and adjust to the desired TSS by either diluting or adding return activated sludge • Pour the mixed liquor into a settling cylinder (100 mm external diameter with a vertical scale of 0 to 100 mm) with a 1 rpm DC motor and stirring device • Allow to settle quiescently out of direct sunlight for 30 min. • After 30 min., record volume occupied by settled sludge	$SSVI = \frac{\text{settled sludge volume (ml/L)}}{\text{MLSS (g/L)}}$ • Initial concentration of 3.5 g TSS/L typically used • TSS concentration in the supernatant at the end of the test quantifies the FSS
Diluted SVI (DSVI)	• Set up several 1-L graduated cylinders • Using well clarified secondary effluent, prepare a series of two-fold dilutions of the mixed liquor (i.e no dilution, 1:1 dilution, 1:3 dilution) • Stir the graduated cylinders for 45s, using a plunger to re-suspended and uniformly distribute the solids • Allow to settle quiescently out of direct sunlight for 30 min. • After 30 min., record volume occupied by settled sludge (SV_{30})	$DSVI\ (mL/g) = \frac{SV_{30}\ (ml/L) * 2^n}{TSS\ (g/L)}$ Where: • SV_{30} is the first dilution where the settled sludge volume is equal to or less than 200 ml/L • n is the number of two-fold dilutions required to obtain SV_{30} <200 mL/L • TSS concentration in the supernatant at the end of the test quantifies the secondary effluent suspended solids (ESS)

12.2.5 Use of Selectors

One method to prevent the proliferation of filamentous organisms in a suspended growth process is to use a selector zone. The purpose of selectors is to provide a food-to-microorganism ratio (F/M_v) that promotes the growth of microorganisms that settle well (MOE, 2008; Metcalf & Eddy, 2003). There are three types of selectors: aerobic selectors; anoxic selectors (typically utilized when a process is designed to nitrify); and anaerobic selectors. Selectors can be implemented within existing bioreactors by:

- installing a partition wall(s);
- installing baffles;
- by blanking off a section of aerators; and/or
- turning off aeration to sections of the bioreactor.

Table 12-10 presents information on the design and operation of various types of selectors. For more information on selectors refer to MOE (2008) and Metcalf & Eddy (2003).

Table 12-10 - Selector Types and Descriptions

(Adapted from MOE, 2008)

Selector Type	Description	Other Information
Aerobic	Three compartment selector recommended with the optimal F/M_v within each compartment as follows: 1st compartment F/M_v= 24 d^{-1} 2nd compartment F/M_v = 12 d^{-1} 3rd compartment F/M_v = 6 d^{-1}	• DO of 1 to 2 mg/L should be maintained • Specific oxygen uptake rate (SOUR) ≥ 65 to 85 mg O_2/g MLVSS/hr
Anoxic	The most efficient design of selector contains three compartment configurations with F/M_v as follows: 1st compartment F/M_v= 12 d^{-1} 2nd compartment F/M_v = 6 d^{-1} 3rd compartment F/M_v = 3 d^{-1}	• DO < 0.5 mg/L should be maintained • Mixing within compartments should be either by mechanical mixers or low rate aeration • If denitrification is desired, a portion of the mixed liquor is recycled to the anoxic zone and adequate nitrate levels must be maintained to ensure stable anoxic operation • One compartment selector with an F/M_v of 0.5 to 1 d^{-1} is adequate to prevent filamentous organisms
Anaerobic	The selector hydraulic retention time should be between 0.75 and 2.0 hours and can be divided into three compartments with similar F/M_v ratios as for anoxic compartments outlined above.	• Mixing within compartments should be by mechanical mixers that do not create excessive turbulence or entrain air

12.2.6 Return Activated Sludge Chlorination

Another method to control sludge foaming and sludge bulking caused by filamentous microorganisms is to dose chlorine to the RAS line. RAS chlorination reduces the concentration of the filamentous microorganisms that are causing the foaming or bulking. Chlorination can result in turbid effluents until the majority of the filamentous microorganisms have been removed.

RAS chlorination can be implemented as a temporary emergency measure or for long term preventative dosing (Chandran *et al.*, 2003). The chlorine dose requirement varies depending on the application and optimal dosing should be determined for each process on site using pilot and full scale testing. From studies conducted at a New York City WPCP, a chlorine dose of 10 to 12 mg Cl_2/g MLSS was effective in bringing foaming under control in emergency applications whereas, for preventative use, a lower chlorine dose of 4 to 6 mg Cl_2/g MLSS dosing was effective (Chandran *et al.*, 2003).

12.3 BIOMASS INVENTORY CONTROL (SOLIDS RETENTION TIME CONTROL)

12.3.1 Purpose and Impact on Process Performance

Controlling the concentration of the biomass within a biological process is typically achieved by managing the solids retention time (SRT) within the bioreactor. SRT is a measure of the length of time that solids are kept within the biological process. SRT is also referred to as the sludge age or the mean cell retention time (MCRT) and is represented in Equation 12.1 below.

$$\text{SRT (days)} = \frac{\text{Total mass of solids within bioreactor(s)}}{\text{Total mass of solids leaving the process daily}} \qquad (12.1)$$

The SRT relates directly to F/M_v ratio, with a high F/M_v correlating to a short SRT and a low F/M_v correlating to a high SRT. Sludge yield can also be inferred from SRT with a higher sludge yield expected in suspended growth processes with lower SRT and high F/M_v. SRT is considered a key design and operating parameter for all biological processes and can greatly impact the overall performance of the treatment process.

The beneficial impacts of properly managing the biomass inventory control include:

- increasing the capacity of unit processes;
- improving the solids settling characteristics;
- enhancing the effluent quality;
- potentially achieving nitrification without requiring expansion of the bioreactor;
- reducing aeration energy required;

- decreasing the amount of solids produced requiring processing; and
- stabilizing operation and minimizing process upsets.

Controlling the solids concentration within biological processes is required to maintain the optimum system performance in terms of both F/M_v ratio and solids separation. Within most biological processes, it is not practical to control the substrate (i.e. food) loading to the process as this is determined by the influent characteristics. Therefore, to operate at the optimum F/M_v, the solids must be maintained at a desired concentration within the bioreactor.

When the concentration of solids is not maintained within a reasonable range of the desired concentration, system performance can be impacted negatively. If the concentration of microorganisms is too low, there may not be an adequate concentration of organisms present to biologically remove the constituents in the sewage to the required levels. Conversely, if the concentration of solids exceeds the desired range, poor settleability or overloading of the solids separation process can result in a decrease in effluent quality.

Further information on solids inventory control is provided in FCM and NRC (2004).

12.3.2 Suspended Growth Processes

Control of the solids concentration within suspended growth processes is accomplished by routine (i.e. continuous or at least daily) wasting of excess sludge, usually from the RAS line. Wasting from the RAS line results in a higher sludge concentration which can improve the operation of downstream sludge treatment processes. Wasting of solids can also take place from the aeration tank where the concentration of solids is uniform. Typical SRT and MLSS values for various suspended growth processes are presented in Table 12-11.

Table 12-11 – Suspended Growth Process Design Parameters

(Adapted from MOE, 2008)

Treatment Process	F/M_v (d^{-1})	MLSS Concentration (mg/L)	Solids Retention Time (SRT) (Days)
Conventional Activated Sludge without Nitrification	0.2-0.5	1000-3000	4-6
Conventional Activated Sludge with Nitrification	0.05-0.25	3000-5000	> 4 at 20 ^oC > 10 at 5 ^oC

Table 12-11 – Suspended Growth Process Design Parameters (continued)

(Adapted from MOE, 2008)

Treatment Process	F/M_v (d^{-1})	MLSS Concentration (mg/L)	Solids Retention Time (SRT) (Days)
Extended Aeration	0.05-0.15	3000-5000	> 15
High-Rate	0.4-1.0	1000-3000	4-6
Contact Stabilization	0.2-0.5	1000-3000	4-10

12.3.3 Fixed Film Processes

Biomass control in fixed film processes is more difficult than in suspended growth processes as the solids are attached to a support media of some type. The SRT within fixed film processes are typically much higher than in suspended growth processes. In fixed film processes, the system self-regulates the concentration of biomass. Biomass inventory is regulated by biofilm detaching on a regular basis from the media depending on the flow patterns and specific conditions on each piece of media. The biofilm that sloughs from the media is removed in the downstream solids separation stage. Generally, when the F/M_v (food-to-biomass ratio in the fixed film) within the biological process increases, the biomass concentration also increases resulting in a thicker biofilm. At a threshold point, the biofilm reaches a maximum thickness, at which point part of the biofilm detaches from the media. When the F/M_v ratio decreases, the concentration of biomass will decrease and will become endogenous. Long periods of low F/M_v operation could result in a thinner biofilm.

The control of fixed film biomass using external means such as chemical, mechanical or by aeration is not recommended over the long term as it can result in the biofilm emitting a larger amount of extracellular polymeric substances (EPS) which can minimize the biofilm's ability to uptake both dissolved oxygen and substrate.

As the operating SRT is difficult to determine and calculate for fixed film processes, the operation is typically based on a loading rate per specific amount of media (i.e. surface area or volume).

12.4 OPTIMIZATION TO ACHIEVE NITRIFICATION

12.4.1 Purpose and Impact on Process Performance

Optimization of a biological process to achieve nitrification may be required if an effluent ammonia limit is applied to the process or a non-toxic effluent is required. Nitrification requires that a two-step biological process takes place including conversion of ammonia through oxidation to nitrite followed by nitrite oxidation to nitrate. As the bacteria responsible for nitrification within biological processes grow much slower than the heterotrophic bacteria that are responsible for carbonaceous BOD removal, the HRT and SRT within nitrifying reactors is usually greater than those required to treat carbonaceous BOD alone.

Determination of the rate of nitrification within a bioreactor can be utilized to ensure that simulations developed accurately predict the nitrification taking place in a specific bioreactor (Chapter 6). The specific nitrification rate can determined using batch experiments in which a sample of biomass is taken and then spiked with ammonia. The decrease in ammonia and increase in nitrate concentration is measured over time while ensuring that oxygen and alkalinity are not limited during the test. Further information on determining the nitrification rate can be found in Melcer *et al.* (2003).

Optimization of a biological process to nitrify involves ensuring that the conditions are appropriate to promote the growth of nitrifying bacteria. A number of environmental conditions can lead to lower than expected nitrification within sewage treatment plants including (Metcalf & Eddy, 2003):

- Low SRT within bioreactor (i.e. less than 4 days at 20 ^{o}C and 10 days at 5 ^{o}C for conventional activated sludge processes);

- Low DO concentration through bioreactor (DO should be greater than 1 mg/L);

- pH outside of the optimal range for nitrification (optimal pH range is 7.5 to 8);

- Elevated concentrations of potentially inhibitory chemicals (e.g. solvent organic compounds, amines, proteins, tannins, phenolic compounds, alcohols, cyanates, ethers, carbamates and benzene);

- High concentrations of metals (i.e. greater than 0.1 mg/L of copper, 0.25 mg/L of nickel, 0.25 mg/L of chromium);

- Low operating temperatures (< 10 ^{o}C); and

- High concentrations of un-ionized ammonia (>100 mg/L) and/or un-ionized nitrous acid (>20 mg/L).

Further information on improving nitrification can be found in Environment Canada (2003).

12.4.2 SRT Control

Because nitrifying microorganisms are slower growing than heterotrophic microorganisms, careful SRT control within bioreactors requiring nitrification is essential. More information on optimizing SRT control can be found in Section 12.3 as well as in Environment Canada (2003) and FCM and NRC (2004).

12.4.3 Upgrading with Fixed Films

One method of optimizing or improving the performance of a suspended growth biological process to achieve nitrification is to upgrade it with a fixed film process (i.e. converting it into a hybrid process). Adding fixed film support media to the aeration basin potentially increases the concentration of biomass within the process which decreases the F/M_v and increases the SRT without dramatically altering the process layout. As the growth rate of the microorganisms that are required for nitrification is much slower than those required for carbonaceous BOD removal and the SRT of fixed films are much longer than that of activated sludge, the addition of media can enhance nitrification within the process. The amount of media required will depend on the type of media selected and the specific nitrification rate. More information on optimization of fixed film systems can be found in Section 12.8.

12.5 BIOLOGICAL NUTRIENT REMOVAL

12.5.1 Purpose and Alternative Process Configurations

Biological nutrient removal (BNR) processes achieve nitrogen and/or phosphorus removal biologically. To remove nutrients biologically, environmental conditions must be controlled to promote the growth of microorganisms capable of removing nitrogen and/or phosphorus. Depending on the configuration utilized, BNR processes involve aerobic, anoxic and/or anaerobic zones which allow the release, uptake and/or ultimate removal of nutrients. A number of process configurations have been developed to promote the environmental conditions that are required to remove nitrogen only, phosphorus only or both. Step processes including step-feed BNRs and step Bio-P can be utilized in order to optimize BNR processes. Modelling as discussed in Chapter 6 should be undertaken prior to altering a process configuration to include step-feed in order to determine how the sewage treatment plant may react. Step feeding bioreactors, as discussed in Chapter 7, can also be utilized to minimize the impacts of wet weather events on effluent quality. Figure 12-4 presents an overview of the common BNR processes.

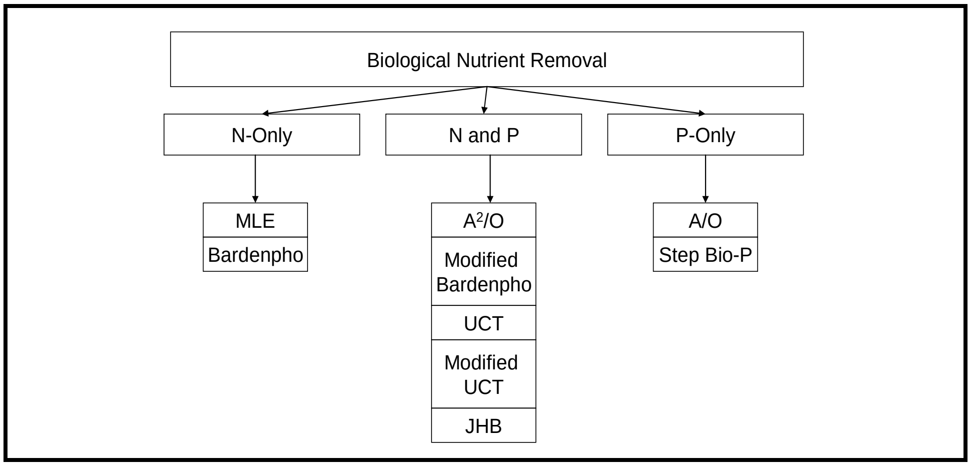

Figure 12-4 – Summary of Biological Nutrient Removal Processes

Implementing biological nutrient removal can be achieved by partitioning existing tanks, blanking aerators, building new tanks and/or installing recycle lines in order to develop the required biological zones. Further information on implementing BNR processes can be found in Section 12.5.2.

As illustrated in Figure 12-4, there are several biological nutrient removal processes including Modified Ludzack-Ettinger (MLE), Bardenpho, A^2/O, Modified Bardenpho, University of Cape Town (UCT), MUCT, Johannesburg (JHB), A/O, and step Bio-P processes. Additional information on biological phosphorus removal can be found in Chapter 16. Supplementary information on biological nutrient removal processes information can be found in WEF/ASCE/EWRI (2005) and Metcalf & Eddy (2003).

12.5.2 Implementing Biological Nutrient Removal

Implementing biological nutrient removal in an existing system requires that the system capacities are well understood. As discussed in Chapter 6, modelling of the system can be used to determine the most appropriate and feasible process configuration. Within conventional activated sludge processes, alterations required to the existing system to develop anoxic and/or anaerobic zones could include:

- turning off sections of aerators or physically blanking off the diffusers if all diffusers are connected to the same air header;
- construction of baffle walls or curtains;
- installation of new piping and pumping systems; and/or
- construction of new tanks.

Optimization of BNR processes might also require the addition of external carbon sources to ensure that efficient denitrification can occur. Additional carbon may be required if there is inadequate soluble BOD or easily biodegradable organics matter present in the influent sewage or within the anoxic zone utilized for denitrification. The addition point of the carbon source depends on the process configuration but usually is dosed to the anoxic zone in two or three stage BNR processes, or to the post-anoxic zone in four or five stage BNR processes. The most common carbon source used is methanol. A mass dosing ratio of methanol to nitrate of 3:1 should be adequate to achieve the desired level of denitrification; however, each system is different and carbon requirements may be higher (WEF/ASCE/EWRI, 2005). Further information on supplemental carbon dosing to achieve denitrification can be found in WEF/ASCE/EWRI (2005).

For BNR processes which include a denitrification step prior to secondary clarification, re-aeration may be required to avoid rising sludge in the clarifiers (Chapter 14).

12.6 BIOREACTOR HYDRAULICS

An understanding of the hydraulics within a bioreactor can be crucial to optimizing the performance of biological treatment systems. Hydraulics in a bioreactor can be impacted by a number of factors including:

- temperature differences within the bioreactor which can cause currents and short-circuiting of flow through a reactor;
- circulation patterns impacted by wind in open tanks;
- insufficient mixing energy in the bioreactor causing zones without adequate mixing; and/or
- reactor design.

12.6.1 Complete Mix Bioreactors

In a complete mix bioreactor, the organic loading, solids concentration, oxygen demand and oxygen availability are uniform throughout the reactor. A complete mix bioreactor is illustrated in Figure 12-5. Complete mix bioreactors are typically mixed using air and optimization of aeration systems is discussed in detail in Chapter 13. Hydraulic and process modelling as discussed in Chapter 6 can be used to estimate the effect of bioreactor configuration on the expected effluent quality and possible methods that can be used to improve the hydraulics. Tracer testing as discussed in Section 12.9.1 can be used to identify short-circuiting or dead-zones within the bioreactor.

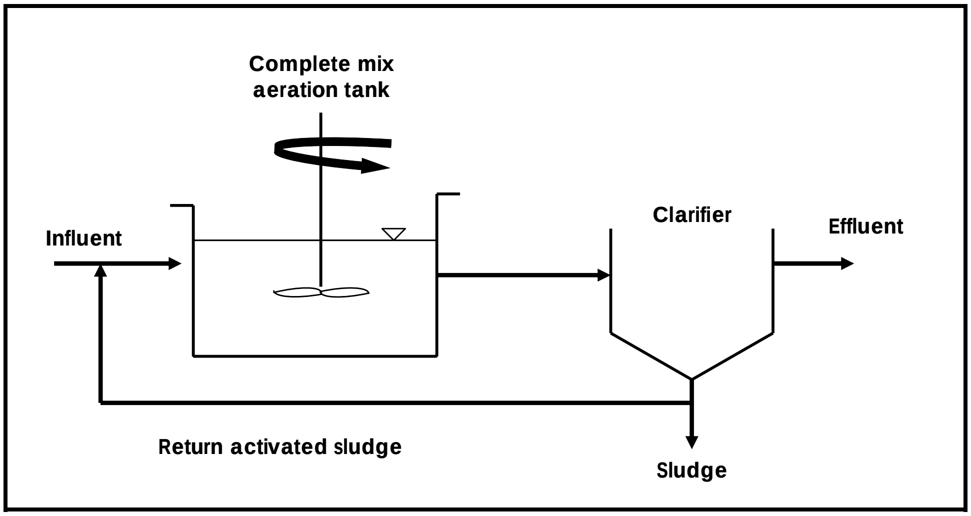

Figure 12-5 - Complete Mix Process Schematic

12.6.2 Plug Flow Bioreactors

Ideal plug flow conditions within a bioreactor occur when a pulse of sewage enters the bioreactor and moves through the reactor without mixing with the plugs of sewage introduced before or after it, as illustrated in Figure 12-6. Plug flow reactors have larger length to width ratios (greater than 4:1) in comparison to complete mix bioreactors (1:1 to 3:1) (MOE, 2008). Baffling and partitioning of the tank can be used to ensure that plug flow conditions are achieved within the bioreactor. In addition, implementation of step-feeding can be utilized in plug flow processes in order to increase the hydraulic capacity of the bioreactor especially during wet weather events (Chapter 7).

To determine the actual flow conditions within the bioreactors and the most effective ways to improve the hydraulics in a plug flow bioreactor, tracer studies (Section 12.9.1) and/or hydraulic modelling (Chapter 6) can be undertaken.

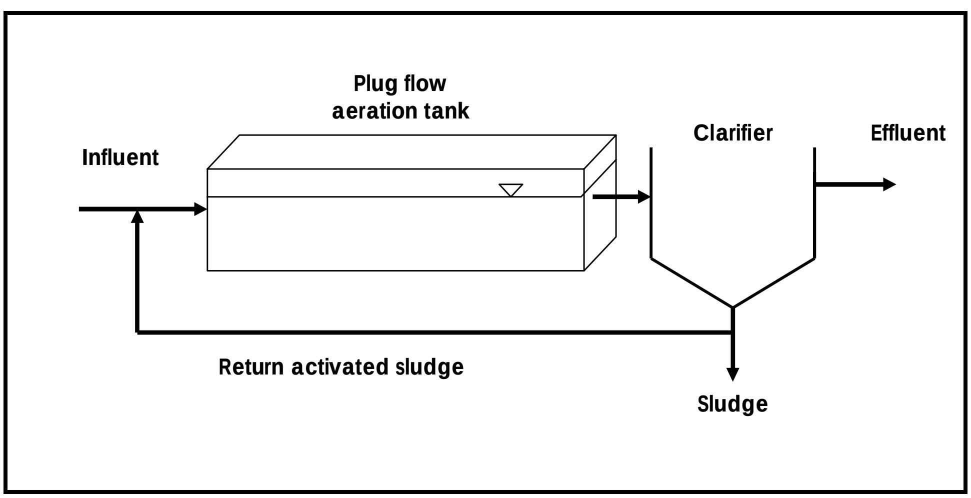

Figure 12-6 - Plug Flow Reactor Process Schematic

Plug flow reactors will produce a lower effluent ammonia concentration than complete mix reactors at the same HRT and SRT operating conditions. Figure 12-7 compares the predicted nitrification performance using a model of a complete mix system with a plug flow system.

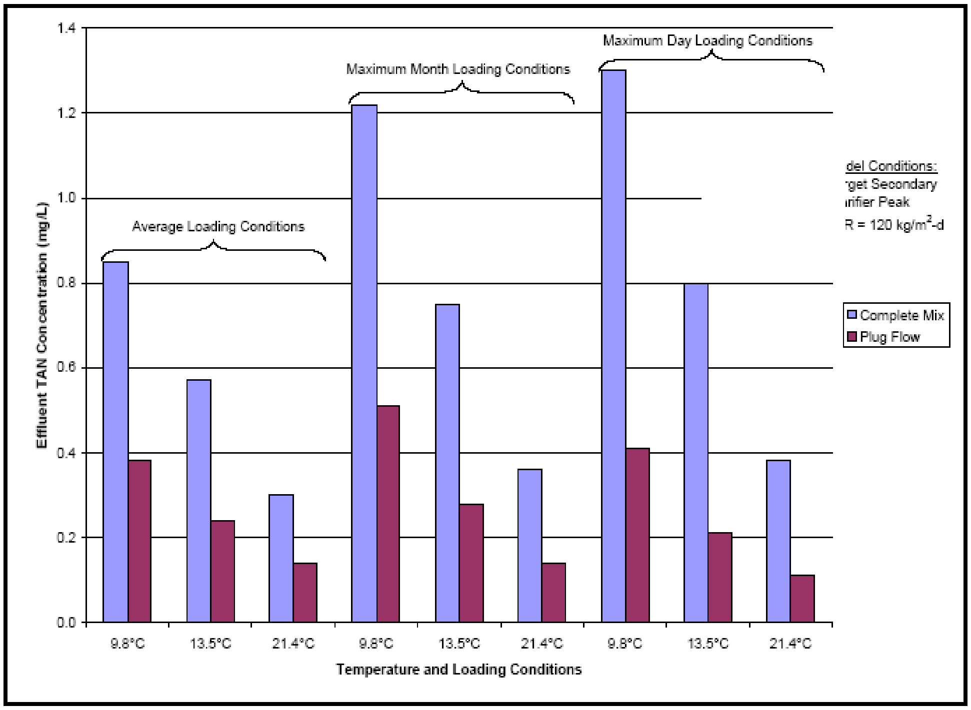

Figure 12-7 - Comparison of Complete Mix and Plug Flow Reactor Effluent TAN Concentrations

12.7 OPTIMIZATION OF SEQUENCING BATCH REACTORS

12.7.1 Cycle Time

In SBRs, the three cycles that can be optimized are the aeration, settling and decanting cycles. A typical cycle time for each cycle of the SBR process is presented in Table 12-12; however, cycle times can vary depending on influent characteristics, reactor design, and operating temperature among other factors.

Table 12-12 – SBR Typical Cycle Times

(Adapted from Metcalf & Eddy, 2003)

SBR Cycle	Cycle Time (hrs)
Fill	3
Aeration	2
Settle	0.5
Decant	0.5

Aeration Cycle Time

Optimization of the duration of the aeration step is crucial to ensure that there has been adequate time of contact between the mixed liquor and substrate in order to oxidize or break down the bio-degradable organics in the wastewater.

To determine the optimal aeration time, a series of samples can be taken during the aeration cycle to establish the reaction rates. If the results indicate that the effluent concentration requirements are met well before the end of the aeration period, the aeration cycle time could be decreased. Alternately, process modelling can be used to optimize the aeration cycle time (Section 6.4.3).

Further information on optimizing aeration systems including on-off air cycling to reduce energy use can be found in Chapter 13.

Settling Cycle Time

Optimization of settling time will ensure that effluent decanting does not remove entrained suspended solids into the effluent. More information on monitoring and optimizing settling within suspended growth process can be found in Section 12.3.2.

Decanting Cycle Time

Optimization of the decanting cycle time or decanting rate may be required if there is evidence of high solids concentrations in the effluent that was not evident in samples taken from the supernatant at the end of the settling cycle. If this is the case, the decanter operation and location should be reviewed. Assuming the decanting point is appropriate, the decanting rate of the effluent could be causing turbulence, resulting in re-suspension of the settled solids.

12.7.2 SRT Control

SRT control within SBRs is typically accomplished by wasting of solids during the decant cycle or during the idle cycle. Information on the control of SRT within suspended growth processes can be found in Section 12.3.1 and 12.3.2 as well as in the FCM and NRC (2004).

12.7.3 Nitrogen Removal

Optimizing SBR processes to achieve or improve nitrogen removal can involve the addition of mechanical mixing during the fill step to enhance the contact between the biomass and influent. Once the fill step is complete, the mechanical mixing can be continued prior to aeration for a period of time to provide a pre-anoxic step. The addition of this step improves nitrogen removal along with solids settling characteristics (Metcalf & Eddy, 2003). Further information on optimizing nitrogen removal within suspended growth processes can be found within Section 12.4.

The implementation of on-off air cycling, as discussed in Chapter 13, can also lead to improvements in both nitrogen removal and solids settling characteristics.

12.8 OPTIMIZATION OF FIXED FILM SYSTEMS

12.8.1 Fixed Film Processes

Like suspended growth processes, fixed film processes require adequate biomass as well as contact time between the biomass and the sewage to ensure optimal performance. Optimization of fixed film processes is more difficult than optimization of suspended growth processes as there is less capability to control the SRT in these processes. Potential mass transfer limitations are also a consideration. Optimization in order to improve treatment capacity of fixed film processes during peak flow events (i.e. wet weather) can be achieved by operating staged fixed film processes in parallel. Further information on alternative configurations during wet weather flow can be found in Environment Canada (2003).

Both hydraulic and biological modelling can be used to assist in determining a system's capacity and the effluent characteristics expected. Improving the biomass and wastewater contact can be accomplished by ensuring that:

- the flow distribution through the process is uniform and no short circuiting is occurring in any region of the reactor (Section 12.6 and Chapter 6);
- the system is not over-loaded either hydraulically or organically (Section 12.1.3); and
- if applicable, any mixing required in the system is consistent and homogeneous throughout the bioreactor.

Optimal fixed film system performance also requires that there is adequate solids separation capacity available to remove the solids which slough from the media. Further information on solids separation can be found in Chapter 14 and Chapter 15.

12.8.2 Integrated Fixed-Film Activated Sludge (IFAS) Systems

Optimization of IFAS processes involves optimizing both the fixed film and suspended growth components of the process as outlined throughout this chapter. Modelling of the hydraulic and biological processes can assist in identifying aspects of the process which may require optimization.

12.9 FIELD INVESTIGATIONS

12.9.1 Tracer Testing

Short-circuiting and dead zones can impact the performance of a bioreactor. Tracer testing can provide an understanding of the actual hydraulic conditions within the reactor.

Tracer testing involves adding a slug of dye or chemical to the inlet of the reactor and then collecting grab samples of the effluent (Daigger and Buttz, 1998). A plot of the effluent concentration of tracer versus time is then prepared to assist with the analysis of the results. In sewage treatment processes, the most commonly used tracers are fluorescein, rhodamine WT and Pontacyle Brilliant Pink B as they can be detected at low concentrations using a fluorometer (Metcalf & Eddy, 2003).

Further information on tracer testing can be found in Chapter 14 as well as in Daigger and Buttz (1998).

12.9.2 Respirometry

Respirometry involves quantifying the biological oxygen consumption rate which directly relates to the condition of the biomass and the substrate removal rate (IWA, 2002). The oxygen consumption rate, also known as the specific oxygen uptake rate (SOUR), is one test that can be useful in indicating the relative biological activity of the microorganisms in the aeration tank (California State University, 1998). Detailed procedures to measure and calculate SOUR can be found in Standard Methods (APHA/AWWA/WEF, 2005).

Typical SOUR values for activated sludge systems are presented in Table 12-13 (California State University, 1998).

Table 12-13 – Typical Specific Oxygen Uptake Rates

(Adapted from California State University, 1998)

SOUR (mg O_2/g MLSS·h)	Indications
< 4	• Biological population is not stable and healthy • Possible toxic load applied to the aeration system
4 – 9	• Over stabilized organic matter • Typical of extended aeration processes • Endogenous respiration activity • Slowly biodegradable organic matter
10 – 20	• Typical of most activated sludge processes
> 20	• Rapidly biodegradable organic matter • Typical of high rate activated sludge processes • May be indicative of under stabilized organic matter

For further information regarding respirometry testing see the IWA Task Group on Respirometry in Control of the Activated Sludge Process (2002) and APHA/AWWA/WEF (2005).

12.9.3 Stress Testing

Stress testing of biological processes can give an understanding of the actual process capacity and capability by operating at loadings beyond the design condition. The results of a stress test can be used to re-rate a bioreactor capacity. Stress testing requires sampling and data collection over a long period of time covering various seasons and a range of load conditions either by sampling naturally occurring high load events (during sustained high load periods and/or storm events) or by artificially increasing the loading to a section of the process (Melcer *et al.*, 2003). Detailed process auditing (Chapter 5) of a sewage treatment plant or bioreactor should be undertaken prior to stress testing. Process auditing can also involve development of a model (Chapter 6) that can be used to give an indication of the sewage treatment plant's response to stress testing conditions. For high load condition testing, sampling and data collection must be carefully planned to ensure that samples are collected frequently enough that peaks in effluent concentrations are not missed.

12.10 CASE HISTORIES

12.10.1 Filamentous Sludge Bulking at the Newcastle WPCP

The following case study is based on information presented in Hansler *et al.* (2006).

The Newcastle WPCP is a CAS process that experienced poor sludge settling from system start-up in 1996. A study was undertaken in order to review the design and operation of the Newcastle WPCP, identify the possible causes of the poor settling and determine remedial action that could be taken in order to improve the sludge settling.

The treatment process at the plant consists of screening, grit removal, one primary clarifier, two activated sludge biological treatment aeration tanks, two secondary clarifiers, and chlorination/dechlorination. There is also a selector zone at the influent of the aeration tanks equipped with fine bubble aeration and jet-mixers. At the start of the study, the RAS was directed to the head of the selector zone and the primary effluent was directed to the main aeration tank. As the plant was operating at 53 percent of design capacity, only one aeration tank and secondary clarifier were operating. The WAS from the bioreactor was co-thickened in the primary clarifier and pumped to a spare aerated grit tank for storage prior to hauling off site for treatment.

Table 12-14 presents a summary of the biological treatment process operating parameters based on 2004 data and a comparison to the typical operating values for a CAS process based on MOE Design Guidelines (MOE, 2008).

Table 12-14–Newcastle WPCP Biological Treatment System Operating Parameters

Parameter	Units	Historical Average	MOE Guidelines (CAS w/ Nitrification)	MOE Guidelines (CAS w/o Nitrification)
Flow	m^3/d	2,170 (53% of design)	-	-
Hydraulic Retention Time (HRT)	hours	9.4	6	6
MLSS	mg/L	1,717	3,000 - 5,000	1,000 - 3,000
MLVSS	mg/L	1,054	-	-
BOD_5 Loading Rate	$kg/m^3{\cdot}d$	0.19	0.31-0.72	0.31-0.72
F/Mv	$gBOD_5$/g MLVSS·d	0.11	0.05-0.25	0.2-0.5
Solids Retention Time (SRT)	days	10-12	> 4 at 20 ^oC >10 at 5 ^oC	4-6

The study identified several approaches to reduce the sludge bulking at the Newcastle WPCP caused by excessive growth of the filament *M. parvicella*. To assess each approach, a systematic testing program was undertaken. Monitoring of parameters included: SVI, settled sludge volume (SSV), microscopic examination, depth of sludge blanket, filament counts and visual indicators (foaming, scum, pin floc and rising sludge blanket). Baseline sampling was conducted to determine the sludge settleability prior to making any process changes. The SVI and SSV for a six month period were reviewed. Over this period, SVI ranged from 258 to 951 mL/g. The SSV during the same period was over 95 percent, indicating little settling.

The operating and/or process modifications that were studied to eliminate filamentous bulking are listed in Table 12-15. For each operational change that was implemented, the process was operated for at least 2 to 3 sludge ages (3 to 4 weeks) in order to fully assess the impact on the plant performance. In most cases, the effect of the operational changes was observed more quickly. The first change made to the process was to discontinue the WAS recycle back to the primary clarifier and send it directly to the sludge holding tank. The scum from the secondary clarifier was also sent directly to the sludge holding tank. Results of this change are shown in Table 12-15 as well as in Figure 12-8.

Table 12-15 – Newcastle WPCP Operational Changes

(From Hansler *et al*., 2006)

Operation/Process Change	Details of Process Change	Result
Divert primary effluent to the selector	• Flow returned to original plant configuration • Primary effluent mixed with the RAS in the selector ahead of the aeration tank	• Effluent TP decreased • SSV decreased from between 30-60% to 20% (Figure 12-8) • Scum still evident • *M. parvicella* still present
Reduce the SRT (increase F/M_v) by lowering the MLSS concentration	• MLSS reduced as low as possible without compromising treatment objectives by increasing sludge wasting	• During period when MLSS was 1,234 mg/L the SSV of 24% compared to an SSV of 86% when the MLSS increased to 2,788 mg/L (Figure 12-9) • *M. parvicella* still present
Bypass the primary clarifier (increase F/M_v)	• Bypassing of the primary clarifier due to a hydraulic retention time of 15 hours in the clarifier (much higher than recommended operating range of 1.5-2 hrs)	• MLSS of 1,775 mg/L resulted in SSV of 22% and SVI of 122 mL/g (Figure 12-10) • *M. parvicella* present but in reduced concentrations • Less scum visible
Decrease operating band in wet well to equalize flow to the plant	• Lowered level at which the sewage collected in the wet well was pumped to the headworks of the plant to maintain more consistent flow to the process	• Improved plant performance but this change alone not able to control *M. parvicella.*

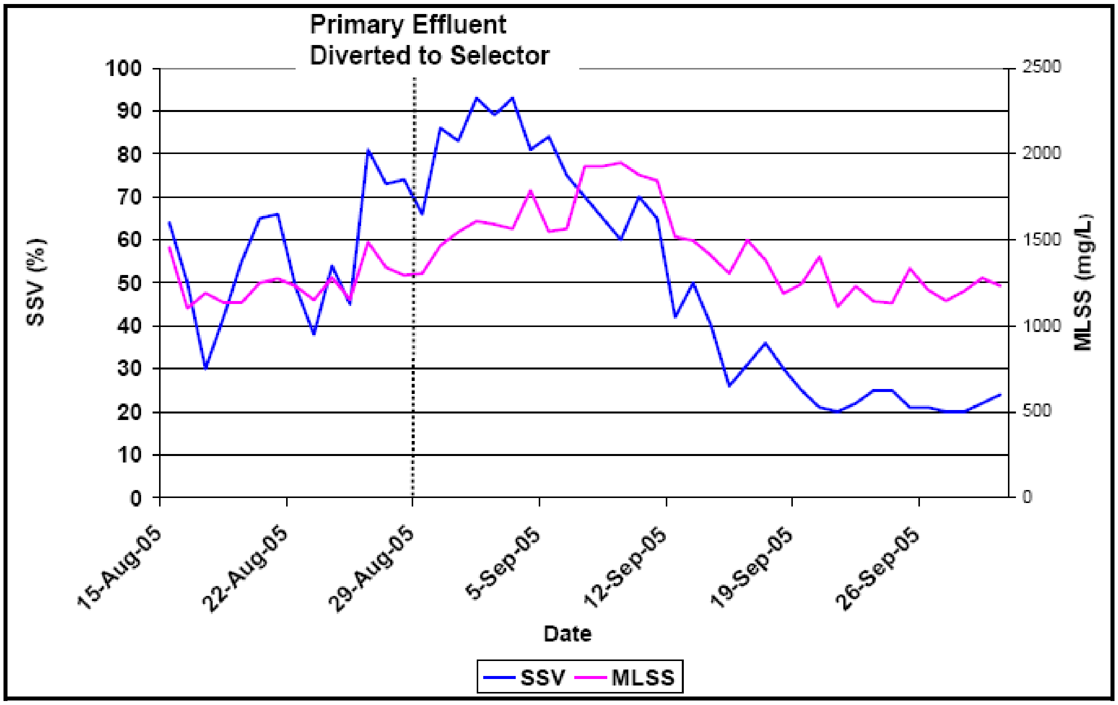

Figure 12-8 – Effect of Diverting Primary Effluent to Selector

(From Hansler *et al*., 2006)

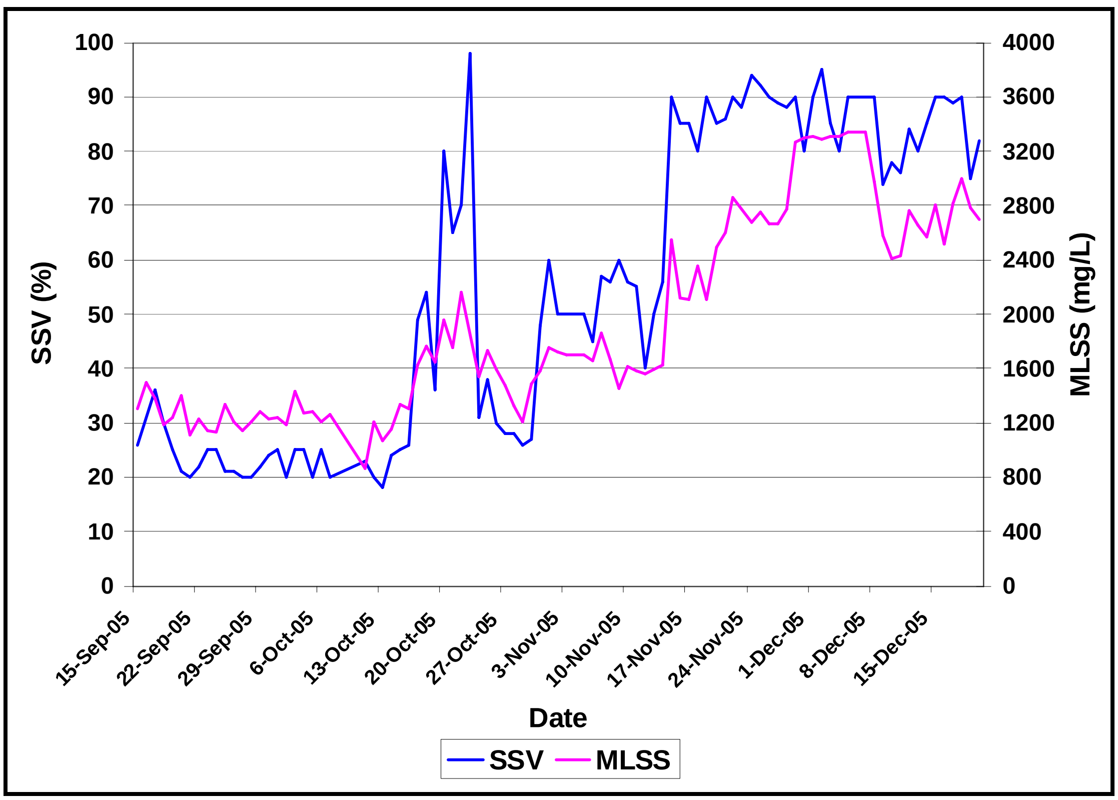

Figure 12-9 – SSV versus MLSS Concentration
(From Hansler *et al*., 2006)

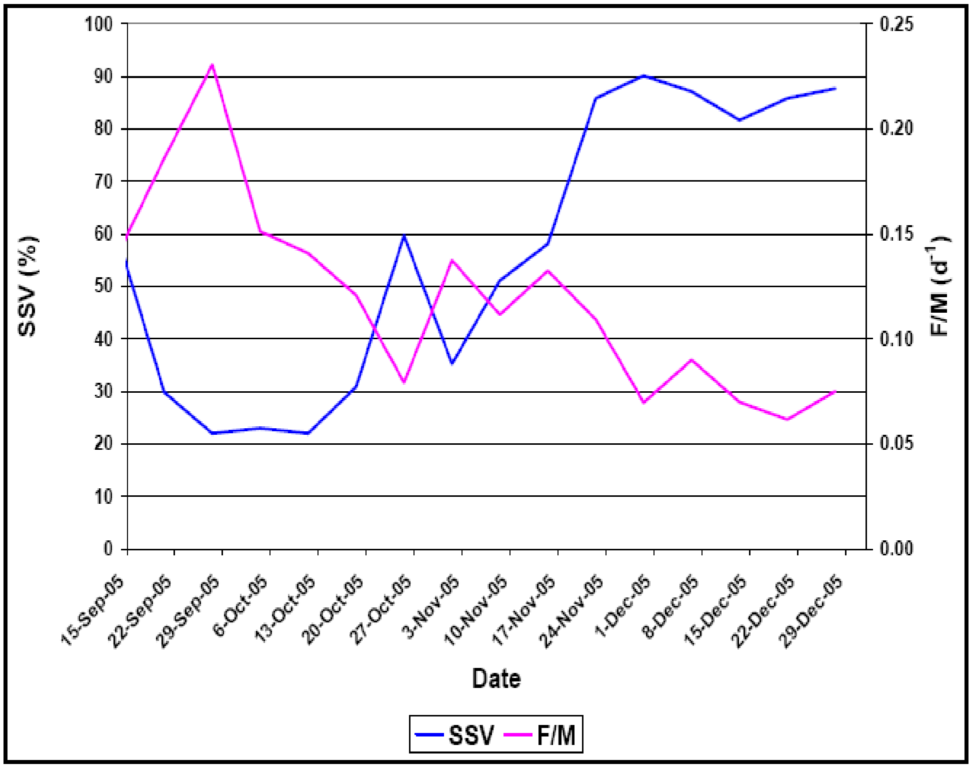

Figure 12-10 – SSV versus F/M Concentration
(From Hansler *et al*., 2006)

A systematic approach to the implementation of a series of operating changes was utilized in order to alleviate the filamentous sludge bulking experienced at the Newcastle WPCP. Through a number of process and operational changes, the sludge settling issues experienced at the plant were resolved.

12.10.2 Integrated Fixed Film Activated Sludge Process Operation at Lakeview WWTP

The following case study is based on information presented in Stricker *et al.* (2007), Maas *et al.*, (2006) and Ross *et al.* (2004).

The Lakeview WWTP in the Region of Peel has been operating a full scale demonstration of the IFAS process with the objective to determine if the technology is capable of achieving nitrification year-round. The IFAS demonstration involved retrofitting one train of the WWTP while a control train was left operating as a CAS process. This allowed for a direct comparison between the IFAS and CAS processes.

The Lakeview WWTP consists of preliminary treatment, primary settling with optional polymer dosage, secondary treatment by conventional activated sludge in plug flow bioreactors, iron dosing for phosphorus co-precipitation, final clarification, and chlorine disinfection. The Lakeview WWTP consists of three parallel plants. The IFAS train was located in one of the four trains of Plant 1.

The CAS and IFAS trains combined treated approximately six percent of the total plant flow.

Figure 12-11 illustrates the layout of the CAS and IFAS trains involved in the demonstration study. Both trains were designed the same originally with a 3,456 m^3 plug flow bioreactor divided into three passes with fine bubble tapered aeration followed by a rectangular clarifier. Ferrous chloride ($FeCl_2$) injection for phosphorus removal occurs at the start of pass 2.

In the IFAS train, the first 25 percent (288 m^3) of the pass was retrofitted to be an anoxic selector to minimize filamentous growth. In the last two passes, the aeration capacity was doubled with the average diffuser density increased from 8.5 percent to 16.8 percent in order to increase the oxygen transfer as well as the mixing capacity within the IFAS passes. In order to contain the carriers/media, these passes were further subdivided into two cells (Figure 12-11) and retrofitted with flat screen and a coarse bubble airknife at the downstream end. The carrier media added was 21 mm in diameter with a length of 16 mm. The media were added at a 46 percent fill ratio which translates to a specific surface area of 185 m^2/m^3 for the IFAS portion of the reactor.

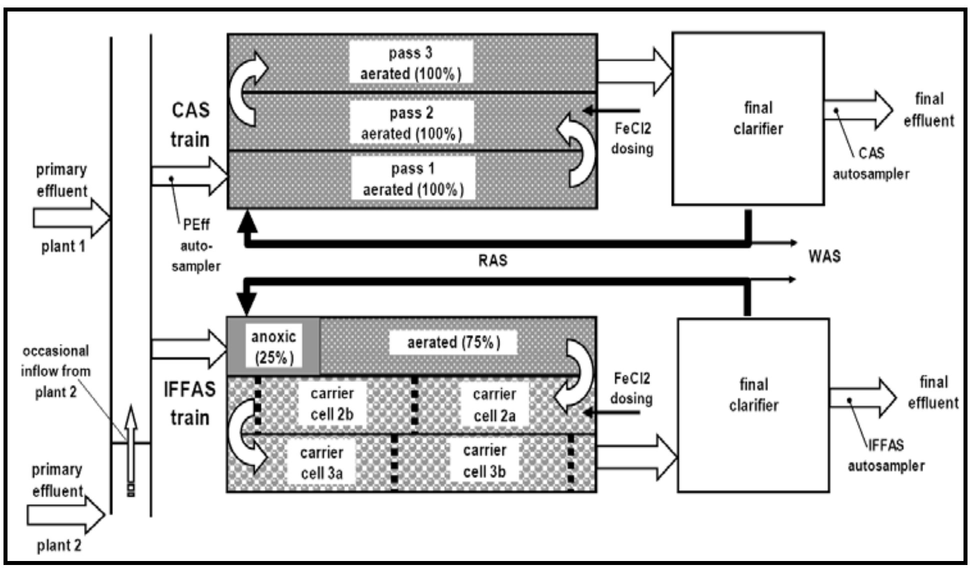

Figure 12-11– Layout of CAS and IFAS Trains at Lakeview WWTP

(From Stricker *et al.*, 2007)

Tables 12-16 to 12-18 present the results of the direct comparison of the operation of the CAS and IFAS trains over an 18-month operating period. Table 12-16 summarizes the influent loadings to the CAS and IFAS processes. Table 12-17 presents the operational parameters for both the CAS and IFAS trains. Table 12-18 presents the effluent concentrations for a number of parameters.

Table 12-16 – Median Values of Influent Loadings (Primary Effluent) to Both Trains

(Adapted from Stricker *et al.*, 2007)

Parameter	Units	Train	Median Value
Flow	m^3/d	CAS	11,425
		IFAS	11,334
TSS	kg/d	CAS	2,085
		IFAS	2,049
BOD_5	kg/d	CAS	2,298
		IFAS	2,355
TKN	kg/d	CAS	496
		IFAS	506
Alkalinity	kg $CaCO_3$/d	CAS	2,890
		IFAS	2,957

Table 12-17 – Median Values of Operational Parameters for Both Trains

(Adapted from Stricker *et al.*, 2007)

Parameter	Units	Train	Median Value
MLSS	mg/L	CAS	2,540
		IFAS	2,640
MLVSS:MLSS	-	CAS	0.73
		IFAS	0.74
Dynamic Suspended SRT	d	CAS	3.6
		IFAS	3.7
Dynamic Total SRT	d	CAS	3.6
		IFAS(1)	6.0
F/M_v	kg BOD_5/ (kgMLVSS·d)	CAS	0.37
		IFAS(1)	0.24
Mixed Liquor Temperature	^oC	CAS(2)	-
		IFAS	20.5
Total Airflow	m^3/h	CAS	10,117
		IFAS	12,771
DO at End of Pass 3	mg/L	CAS	5.2
		IFAS	7.0

Table 12-17 – Median Values of Operational Parameters for Both Trains (continued)
(Adapted from Stricker *et al.*, 2007)

Parameter	Units	Train	Median Value
Final Clarifier SLR	kg/(m^2.d)	CAS	98
		IFAS	113
RAS/WAS TSS Concentration	mg/L	CAS	5950
		IFAS	5230
SVI	mL/g	CAS	86
		IFAS	113
Notes: 1. Calculations include the fixed biomass for the IFAS. 2. Temperature was only measured in the IFAS train and assumed to be the same in the CAS train.			

Table 12-18 – Median Values of Final Effluent Concentrations for Both Trains
(Adapted from Stricker *et al.*, 2007)

Parameter	Units	Train	Median Value
TSS	mg/L	CAS	7
		IFAS	5
COD	mg/L	CAS	55
		IFAS	55
$CBOD_5$	mg/L	CAS	4
		IFAS	<2
TAN	mg/L	CAS	11.0
		IFAS	3.5
NO_2-N + NO_3-N	mg/L	CAS	9
		IFAS	15
Alkalinity	mg $CaCO_3$/L	CAS	113
		IFAS	74

During the 18-month study, the two trains operated under similar hydraulic, organic and nitrogen loadings, as well as suspended biomass (MLSS, suspended SRT) and temperature conditions. The IFAS train contained 50 percent more biomass (60 percent within the first cell) and as a result had a lower F/M_v, a higher aeration requirement and a higher total SRT. It should be noted that for the first 11 months of the study, both trains were overloaded as a result of input from side streams from biosolids processing.

Some operational issues were experienced which impeded the performance of the IFAS train including:

- carrier/ media mixing issues;
- winter time sludge foaming;
- issues related to storing and transferring the carrier media during bioreactor maintenance; and
- downstream and upstream carrier break out.

In the CAS train, nitrification activity was not stable and generally lower than that in the IFAS train with a median effluent value of 11 mg TAN/L and it was only able to achieve the TAN objective (5 mg/L) 32 percent of the time over the entire study period. Nitrification in the CAS train improved to 47 percent during periods of operation at or below the design loading (Table 12-18). The poor nitrification capacity was the result of a relatively low SRT which had a median value of 3.6 days.

In comparison, the IFAS train achieved a median effluent TAN concentration of 3.5 mg/L and 58 percent of the time was able to meet an objective of 5 mg/L during high loadings. During the period of operation at or below the design loading the IFAS was able to meet the TAN objective 67 percent of the time. Additionally, it was evident that the IFAS system had enhanced nitrification in comparison to the CAS train during cold weather periods and was able to maintain complete nitrification for three weeks longer than the CAS after the onset of the cold weather operating period.

The nitrate plus nitrite median concentration was 6 mg/L higher in the IFAS train as a result of the higher degree nitrification within that train. The nitrate plus nitrite results would suggest that denitrification is a marginal process in the IFAS train.

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CHAPTER 13

AERATION SYSTEMS FOR BIOLOGICAL TREATMENT

CHAPTER 13

AERATION SYSTEMS FOR BIOLOGICAL TREATMENT

13.1 OVERVIEW OF AERATION SYSTEMS FOR BIOLOGICAL TREATMENT

The presence of oxygen is vital to the aerobic biological treatment of sewage. However, the low solubility of oxygen limits the oxygen transfer across the air-water interface and needs assistance through mechanical or physical means to provide sufficient oxygen to meet the requirements for biological treatment. Sufficient gas-liquid surface area is required for oxygen transfer to satisfy the needs of biological treatment. Aeration systems increase oxygen transfer by increasing the surface area available for mass transfer by the addition of bubbles, or through mechanical mixing.

This chapter focuses on the optimization of aeration systems for biological treatment.

13.1.1 Types of Aeration Systems and Typical Oxygen Transfer Efficiencies

Table 13-1 presents aeration systems commonly used in biological treatment as well as typical clean water standard oxygen transfer and aeration efficiencies. It should be noted that standard oxygen transfer and aeration efficiencies are dependent on the submergence of diffused air systems.

Table 13-1 – Aeration Systems – Systems and Typical Performance
(Adapted from WEF/ASCE, 1998)

Aeration System	Standard Oxygen Transfer Efficiency	Standard Aeration Efficiency (1)	Advantages / Disadvantages
Fine Bubble Diffusers	13 – 45 %	1.9 – 6.6 kg O_2/kWh	• High efficiency • Flexible operation • Potential for clogging
Jet Aerators (fine bubble)	18 – 25 %	2.2 – 3.5 kg O_2/kWh	• Good mixing • Limited geometry • Potential for clogging
Mechanical Surface Aerators	-	1.1 – 2.5 kg O_2/kWh	• Flexible operation • Potential for icing in cold climates
Coarse Bubble Diffusers	9 – 13 %	1.3 – 1.9 kg O_2/kWh	• Resistant to clogging • Low oxygen transfer efficiency

Notes:
1. Based on a submergence of 4.3 m for diffused air systems.

13.1.2 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with aeration systems are shown in Table 13-2.

Table 13-2 – Aeration Systems – Common Problems and Potential Process Impacts

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Uneven aeration.	• Uneven bubbling pattern at surface of aeration tank • Uneven DO concentrations within individual aeration tanks, or between various aeration tanks	• Clogged or broken diffusers and/or air headers (Section 13.2) • Insufficient air flow control (Section 13.3.2)
Insufficient aeration.	• Low DO readings (continually or diurnally during high loading conditions) • Presence of filaments/increased mixed liquor SVI • Septic conditions in aeration basins or secondary clarifiers (i.e. – black mixed liquor, rising black sludge in secondary clarifiers)	• Clogged or broken diffusers and/or air headers (Section 13.2) • Poor DO process control (Section 13.3.2) • Undersized aeration system design • Surfactants in wastewater
Over-aeration.	• DO readings are consistently > 5 mg/L • Foaming • Floc breakup/pin floc as a result of excessive turbulence	• Poor DO process control (Section 13.3.2)

13.2 AERATION SYSTEM MAINTENANCE

Over time, the efficiency of aeration systems can be reduced by diffuser clogging, reduced mechanical efficiency, and general wear and tear. Regular cleaning and maintenance serves to avoid these issues and to maintain optimal operation of the system.

Equipment supplier cleaning and maintenance recommendations and schedules should be followed. The practice of regular cleaning and maintenance can prevent decreased system performance, reduced oxygen transfer efficiency (OTE), and premature wear of mechanical components. Regular cleaning and maintenance can reduce the frequency of diffuser clogging in diffused air systems and downtime in mechanically aerated systems.

13.3 UPGRADING FOR ENERGY USE REDUCTION

Oxygen requirements in bioreactors represent the single largest energy requirement in activated sludge facilities, accounting for 50 to 90 percent of a sewage treatment plant's power consumption (WEF/ASCE, 1998).

Energy use is directly related to the size of the aeration system and the efficiency with which oxygen transfer is accomplished. Optimization of aeration equipment to reduce energy consumption may result in significant operational cost savings for the plant, and are discussed in the following sections.

13.3.1 Measuring Energy Use in Aeration

If separate power metering exists for the aeration system, then energy usage may be obtained from the meter readings. Otherwise, the energy use for aeration may be estimated based on blower/motor ratings and runtimes. Determining baseline energy consumption values will provide a basis for assessing the impact of energy use optimization measures on energy consumption and associated operational costs.

13.3.2 Dissolved Oxygen Control

Installation of online DO analyzers in the aeration tanks, tied to control loops and programmable logic controllers, can provide automatic DO control and optimize the operation of the aeration system in terms of energy consumption. Blower operation and/or air piping control valve positions can be manipulated to maintain DO set-points within and between bioreactors.

With proper automated controls, the aeration system can reduce the volume of air supplied to the aeration tanks during low loading conditions, thus reducing energy use. In addition, by maintaining the minimum operating DO concentration required to sustain effective biological activity (1.0 to 2.0 mg/L depending on whether nitrification is required) in the bioreactor, continuous or diurnally low DO conditions as a result of insufficient aeration can be prevented, potentially resulting in improved biological process performance. More information regarding the configuration and development of automatic DO control systems can be found in WEF (1997).

Because automatic DO control is dependent on the operation of online DO analyzers, care must be taken to ensure the accurate, consistent and continuous operation of the instrumentation. Manufacturer recommended maintenance should be performed at the prescribed frequency. The Instrumentation Testing Association (ITA) has undertaken testing of online DO analyzers and has considerable information on the performance, accuracy and life-cycle costs of such equipment. This information can be obtained from ITA's website www.instrument.org. Other online instrumentation required for automatic DO control include air flow measurement devices, and pressure and temperature sensors to monitor blower suction and discharge conditions (WEF, 1997).

13.3.3 On-Off Aeration

On-off (also referred to as cyclic or intermittent) aeration can be used to reduce energy consumption during biological treatment. The use of on-off aeration is well suited to the SBR process or flow-through systems where biological nitrogen removal is also required (Chai *et al*., 2006; Habermeyer and Sanchez, 2005; Chen *et al*., 2001).

A treatment plant must be providing nitrification in order to implement on-off aeration (MOE, 2000). This is because nitrates and nitrites, which are produced during the nitrification process, are utilized by microorganisms during the air-off (anoxic) cycle as an oxygen source.

An SRT control program (Section 12.4.2) is required to ensure continued nitrification once the on-off aeration strategy is implemented. For plants with effluent TAN compliance requirements, the impact of implementing on-off aeration on effluent TAN concentrations should be closely monitored, especially during cold weather or increased loading conditions, to ensure continued compliance with effluent limits. In some cases, implementation of an on-off aeration strategy may not be feasible.

The types of aeration systems suitable for an on-off aeration strategy include fine bubble membrane diffusers, coarse bubble diffusers, mechanical aerators, and jet aerators. On-off aeration is not suitable for fine bubble stone or ceramic diffusers, or porous plastic diffusers due to the potential for clogging during the air-off cycle (MOE, 2000). In addition, the impact of air on-off cycles on the operation of the blowers should be evaluated on a case-by-case basis.

In order to successfully operate the aeration system in on-off cycles, the ratio of aeration system uptime and downtime, and the frequency of on-off cycles must be determined empirically through on-site testing. Process control of an on-off aeration strategy can be enhanced through the installation of online DO analyzers to automatically control the duration of the aeration cycles. Supplemental mechanical mixing may be required to maintain mixed liquor in suspension during un-aerated periods. Information regarding techniques for optimizing on-off aeration can be found in MOE (2000).

13.3.4 Higher Efficiency Equipment

Higher efficiency equipment can result in better oxygen transfer and lower aeration costs by as much as 20 to 30 percent (Mace, 2004). Due to improved oxygen transfer characteristics, oxygen is more efficiently supplied to the wastewater, reducing energy costs to provide the required amount of air in comparison to less efficient systems.

Higher efficiency equipment may require more frequent cleaning and maintenance in order to maintain optimal operation and performance.

13.4 FIELD INVESTIGATIONS TO MEASURE OXYGEN TRANSFER EFFICIENCY

13.4.1 Clean Water Tests

Clean water tests are used to determine the standard oxygen transfer efficiency (SOTE) of an aeration system.

The measurement of clean water oxygen transfer rate (OTR) is described in the American Society of Civil Engineers (ASCE) clean water standard (ASCE, 1992). This method consists of the removal of DO from water by the addition of sodium sulphite in the presence of a cobalt catalyst, followed by the subsequent re-oxygenation of the water to near saturation levels. During the re-oxygenation period, DO levels are measured throughout the tank as specified in the procedure.

The data obtained is used to estimate the volumetric mass transfer rate of oxygen in the clean water and the effective standard OTR (SOTR) and standard OTE (SOTE). Standard conditions for oxygen transfer are defined as water temperature of 20 °C, barometric pressure of 101.3 kPa, and DO concentration of 0 mg/L. Standard aeration efficiency (SAE), in kg O_2/kWh, can be calculated by dividing SOTR by the power input (WEF/ASCE, 1998).

13.4.2 Field Oxygen Transfer Testing

In practice, the field oxygen transfer efficiency (FOTE) is less than the SOTE due to characteristics of the wastewater, operating DO concentration in the test tank, wastewater temperature, barometric pressure, tank geometry, and fouling of the diffusers in a diffused air system. The effects of these inefficiencies can be quantified by testing the aeration equipment in mixed liquor in the actual aeration basin where it is installed and applying appropriate correction factors to the SOTE.

Two methods available for field oxygen transfer testing are outlined in the following sections, namely Off-Gas Analysis (Section 13.4.3) and the Hydrogen Peroxide Method (Section 13.4.4).

13.4.3 Off-gas Analysis

Off-gas analysis is a non-interruptive, steady state technique consisting of the measurement of gas entering and exiting the treatment unit. Off-gas analysis provides a means to determine the FOTE of a diffused air system.

As part of the testing procedure, gas exiting the aeration tank is collected by a floating hood and directed to an analyzer. The gas flow is measured and analyzed for oxygen (O_2) and carbon dioxide (CO_2) content. A comparison of the composition of the exiting gas with reference air (which is equivalent to the air entering the aeration tanks) is conducted and used to determine the changes in the composition of the gas as it passes through the aeration tank. Based on the change in composition of the gas, the FOTE of the system can be determined.

Detailed information regarding the testing procedure can be found elsewhere (Daigger and Buttz, 1998).

Off-gas analysis is a point method of analysis that determines the FOTE at specific points in the process tank. This method can be used to determine spatial variations in FOTE.

It should be noted that off-gas analysis requires specialized equipment, is limited in application to diffused air systems, and can be difficult for highly turbulent coarse bubble diffused air systems.

13.4.4 Hydrogen Peroxide Method

Hydrogen peroxide analysis is a non-steady state, interruptive technique that allows the FOTE of any type of aeration system, including mechanical aeration systems, to be determined.

To conduct the test, an aeration basin is taken offline, such that there is no flow through the reactor and the aeration system is turned off. Hydrogen peroxide is then added to the mixed liquor. The microorganisms convert the hydrogen peroxide to oxygen, thereby supersaturating the aeration tank contents with respect to DO. The aeration system is then turned on, and as it is operated, DO is stripped from the aeration tank. The decaying DO versus time trend is used to determine the FOTE for the oxygen transfer system. Detailed information regarding the testing procedure can be found elsewhere (Daigger and Buttz, 1998).

In order to ensure accurate measurements, the aeration basin must be taken offline for the tests, and complete mixing of the hydrogen peroxide must be ensured. Hydrogen peroxide method is relatively expensive due to the quantity of hydrogen peroxide required. Appropriate safety procedures must also be taken during the testing due to the highly reactive nature of hydrogen peroxide.

Hydrogen peroxide analysis is a composite method, measuring the FOTE for the entire process tank. It cannot be used to determine spatial variations in FOTE across the tank.

13.5 CASE HISTORIES

13.5.1 G.E. Booth (Lakeview) WWTF, Mississauga, Ontario – Optimization of Dissolved Oxygen Control

The following case study is based on information presented in Mroczek *et al*. (2008).

Background and Objectives

The G.E. Booth (Lakeview) WWTF is a conventional activated sludge treatment facility with an average rated capacity of 448,000 m^3/d. The liquid treatment train consists of three plants: Plant 1 has no DO control system, and Plants 2 and 3 each have a separate DO control system. Fine bubble diffusers are installed in all three plants. In 2006, the aeration system was upgraded to allow for full nitrification.

The goal of the study was to optimize the existing DO control systems in Plants 2 and 3 to reduce energy use at the G.E. Booth (Lakeview) WWTF. The study was undertaken in the summer of 2007.

Optimization Methodology

A three-stage approach was utilized to optimize the DO control systems.

Stage 1 – Initial Calibration and Observations

The online DO probes were calibrated when the DO control systems were brought online in 2007. Detailed observations of the operation of the aeration systems, including recorded DO levels in the aeration tanks, blower operation, and air header pressures and valve positions were recorded and analyzed to develop a baseline for current system performance.

Stage 2 – Check and Modify System Components

During Stage 2, the results obtained during the Stage 1 baseline assessment were used to identify additional system components that required:

- calibration, such as air flow meters;
- adjustment, such as air valves that should have been in a fully open position yet were discovered in a throttled position; and
- repair, such as air leaks in the air piping.

In addition, the previous calibration of the DO probes were confirmed utilizing hand-held equipment.

Stage 3 – Check and Modify System Controls

Once the condition of the physical components of the DO control system had been confirmed, as part of Stage 2, an intensive analysis of the control loop and programming logic was undertaken so that deficiencies could be identified and rectified.

Several optimization activities were undertaken as part of Stage 3 including:

- Adjustment of the minimum air valve open position set-point from 15 to 5 percent to avoid excessive volumes of air being delivered at minimum valve open conditions;
- Adjustment of aeration tank DO set-points to 2 mg/L along with an increase in the maximum air header pressure set-point to allow for greater control of the number of blowers called to service and maintenance of the DO set-point;

- Adjustment of the influent flow rate to individual aeration tanks to equalize flows, and thus loadings, between the units;

- Adjustment of time delays associated with blower operation to avoid excessive periods of two blowers operating at minimum output conditions. This resulted in energy savings due to shorter periods of blower operation at the minimum, and least efficient, output conditions; and

- Changing the lead-lag configuration of the blowers. As a second blower is called to service, both the lead and lag blowers reduce to minimum output. As oxygen demands increase, the lead blower is ramped up to 100 percent while maintaining the lag blower at minimum output. A further increase in oxygen demand would result in an increase in the output of the lag blower. As a result, energy savings were recognized by setting a newer blower, which is more energy efficient during modulating operation, as the lag blower, while setting an older blower, which is only efficient when operating at or near 100 percent output, as the lead blower.

Results

A significant reduction in operating DO values was observed as a result of the implementation of the optimization program, with average DO values dropping from 6 to 7 mg/L prior to implementation to 2 to 3 mg/L post implementation. Based on preliminary energy consumption data, it was also expected that a blower energy consumption reduction of up to 15 to 20 percent could be realized, however further monitoring would be required to confirm these values.

13.5.2 Implementation and Optimization of On-Off Aeration at Various Ontario Sewage Treatment Plants

The following case study is based on information presented in MOE (2000).

Background and Objectives

Over the period 1997 to 1998, four STPs in Ontario, namely the Cobourg #2 STP, Deseronto STP, Elmvale STP, and Paris STP, took part in a study to evaluate the impact of implementing full-scale on-off aeration strategies.

The objectives of the study were to evaluate the impact of implementing on-off aeration in terms of effluent quality and energy usage.

Summary of Full Scale Results

As part of the study, on-off aeration was implemented in conjunction with SRT control measures to ensure continued nitrification. The trials were run at each facility over 6 to 12 month periods. Information regarding the performance of each facility is outlined below.

Cobourg #2 STP

The Cobourg #2 STP, a conventional activated sludge plant with a design capacity of 11,700 m^3/d, was operating at approximately 50 percent of its design capacity. Treatment was provided in two parallel trains, and each aeration tank was equipped with mechanical aerators.

Flow splits between the two parallel trains were adjusted from 50/50 to approximately 35/65 to account for the difference in aerator capacity in each aeration tank. Cycle times were set to 30 minutes air-on / 30 minutes air-off. Mixing during air-off cycles was provided by residual turbulence and influent flows in one train, while the aerator was set to low speed during the air-off cycle to provide mixing in the second train.

No significant impacts were observed on effluent $CBOD_5$, TSS, or TP concentrations during the study. However, significant reductions in effluent TAN were observed due to the increased SRT control in addition to modest reductions in effluent TN concentrations as a result of denitrification during the air-off cycles.

The cost to implement on-off aeration was less than $1,000 and reduced plant energy costs by $4,000, which is equivalent to 6 percent of the plant's annual energy costs.

Deseronto STP

The Deseronto STP, an extended aeration package plant with a design capacity of 1,400 m^3/d, was operating at approximately 93% of its design capacity. The aeration tank was equipped with fine bubble membrane diffusers.

Cycle times were set to 30 minutes air-on / 30 minutes air-off during the day and 30 minutes air-on / 45 minutes air-off during the night. Mixing during air-off cycles was provided by residual turbulence and influent flows.

No significant impacts were observed on effluent $CBOD_5$, TSS, or TP concentrations during the study. An increase in effluent TAN concentrations during the winter, from less than 2 mg/L to over 12 mg/L, was observed due to a loss of nitrification. Due to sludge handling problems, operations staff reduced the SRT from 20 - 40 days to less than 20 days, resulting in a washout of nitrifiers.

Prior to the implementation of on-off aeration, the blowers accounted for 66 percent if the plant's energy usage. During on-off aeration, this value was reduced to 40 percent, reducing plant energy costs by 21%. The cost to implement on-off aeration was about $2,700 and reduced plant energy costs by approximately $3,500.

Elmvale STP

The Elmvale STP, an extended aeration plant with a design capacity of 1,500 m^3/d, was operating at approximately 73% of its rated capacity. Treatment was provided in two parallel aeration tanks, each equipped with a jet aeration system. Prior to implementing the on-off aeration strategy, one aeration tank was taken offline to improve the energy efficiency of the plant, as it was determined that the aeration tanks were significantly oversized based on their design capacity.

Initially, cycle times were set to 30 minutes air-on / 30 minutes air-off; during the course of the study, these were optimized to 45 minutes air-on / 75 minutes air-off during AM hours, and 60 minutes air-on / 60 minutes air-off during PM hours. Mixing during air-off cycles was provided by hydraulic pumping using the jet pumps.

No significant impacts were observed on effluent $CBOD_5$, TSS, TP, or TAN concentrations during the study. Significant reductions in effluent TN concentrations were observed due to the denitrification provided during the air-off cycles.

The cost to implement on-off aeration was approximately \$8,000 and reduced plant energy costs by about \$27,500, which is equivalent to 45 percent of the plant's annual energy costs.

Paris STP

The Paris STP, an extended aeration plant with a design capacity of 7,100 m^3/d, was operating at approximately 51% of its design capacity. Treatment was provided in two parallel treatment trains, each equipped with two aeration tanks. All aeration tanks were equipped with mechanical surface aerators.

Cycle times were set to 30 minutes air-on / 30 minutes air-off.

No significant impacts were observed on effluent $CBOD_5$, TSS, or TP concentrations during the study. However, nitrification was lost during the winter months due to an increase in industrial loadings to the plant, resulting in DO limited conditions within the aeration tanks. As a result, the on-off aeration strategy was suspended, and the system was returned to full aeration mode.

The cost to implement on-off aeration was approximately \$8,200 and reduced plant energy costs by about \$5,600, which is equivalent to 13 percent of the plant's annual energy costs. However, this strategy was not sustainable during winter months.

Summary

Based on the results from the four STPs, the implementation of on-off aeration was found to be feasible for various types of treatment processes and aeration systems. No impact was noted on effluent $CBOD_5$, TSS or TP concentrations; however, implementation of on-off aeration increases the sensitivity of the nitrification process.

Based on capital costs associated with retrofitting these facilities to provide on-off aeration, which ranged from \$1,000 to \$8,000, and the estimated savings in energy costs, it was estimated that the payback period would range from between 1 to 1.5 years.

13.6 REFERENCES

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CHAPTER 14

SECONDARY CLARIFICATION

CHAPTER 14

SECONDARY CLARIFICATION

14.1 OVERVIEW OF SECONDARY CLARIFICATION

14.1.1 Purpose of Secondary Clarification and Types of Secondary Clarifiers

The purpose of secondary clarification is to provide solids separation of biomass and other solids from the liquid stream downstream of a biological treatment process (either suspended growth, attached growth, or hybrid) to produce a clarified secondary effluent.

Secondary clarifiers can be either circular or rectangular tanks. Baffles are normally installed to promote solids settling by improving the hydraulic conditions within the clarifier. Sludge collection mechanisms are used to remove the sludge that accumulates on the bottom of the tank, while skimmer mechanisms are used to remove scum and other floating objects that accumulate on the liquid surface.

Optimization of the secondary clarification process can involve modifying flow control structures (such as effluent weirs and baffles within the clarifiers) and operational practices (such as RAS pumping methods or chemical dosages) to improve the performance of the secondary clarifiers with respect to solids separation and removal. Because of the high solids concentration of the mixed liquor influent to secondary clarifiers located downstream of suspended growth and hybrid biological treatment processes, the performance of the secondary clarifiers can also be improved by improving the setting characteristics of the solids through upstream operational changes. Optimization of sludge settling characteristics by biological process changes are described in Chapter 12.

14.1.2 Evaluating Process Performance

Table 14-1 presents a typical secondary clarifier monitoring program, in terms of sampling locations and analyses that would be used to evaluate the performance of the process.

Table 14-1 - Secondary Clarification - Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Secondary Clarifier Effluent	Composite Recommended	• $CBOD_5$ • TSS • TP	Although the secondary effluent concentrations of these parameters are affected by the upstream biological treatment process, these values can also provide insight into secondary clarifier performance. If the performance of a particular clarifier is suspect, effluent samples from individual clarifiers can be collected for comparison purposes.
Sludge Blanket	Discrete	• Sludge blanket depth	Commonly accomplished using a “Sludge Judge”, hand-held solids analyzer, or on-line sludge blanket monitor. It is recommended that sludge blanket readings be taken at various longitudinal locations (for rectangular clarifiers) or radial locations (for circular clarifiers) to develop a sludge blanket profile.
Return and/or Waste Activated Sludge	Composite Recommended	• Flow rate of each of RAS and WAS streams • TSS	Composite samples can be collected as a series of grab samples throughout the day. It is recommended that several grab samples be collected at different times during the day so that results are not biased towards operational conditions specific to certain times of day. If possible, RAS flow rates / volumes for each clarifier should be recorded.

In addition to the recommended sample locations and analyses presented in Table 14-1, it is recommended that the following also be monitored:

- Secondary clarifier influent flow;
- Characteristics of secondary clarifier influent. In the case of a suspended growth process, parameters to be recorded include MLSS, SVI and SSVI (Chapter 12);
- Quantity and characteristics of coagulants and/or polymers added upstream of the secondary clarifiers or in the biological treatment process, if applicable; and
- Secondary ortho-phosphorus concentrations if coagulant(s) are being used for chemical phosphorus removal as part of the secondary treatment process.

Figure 14-1 presents a process schematic of a typical secondary clarification process illustrating the various sampling locations.

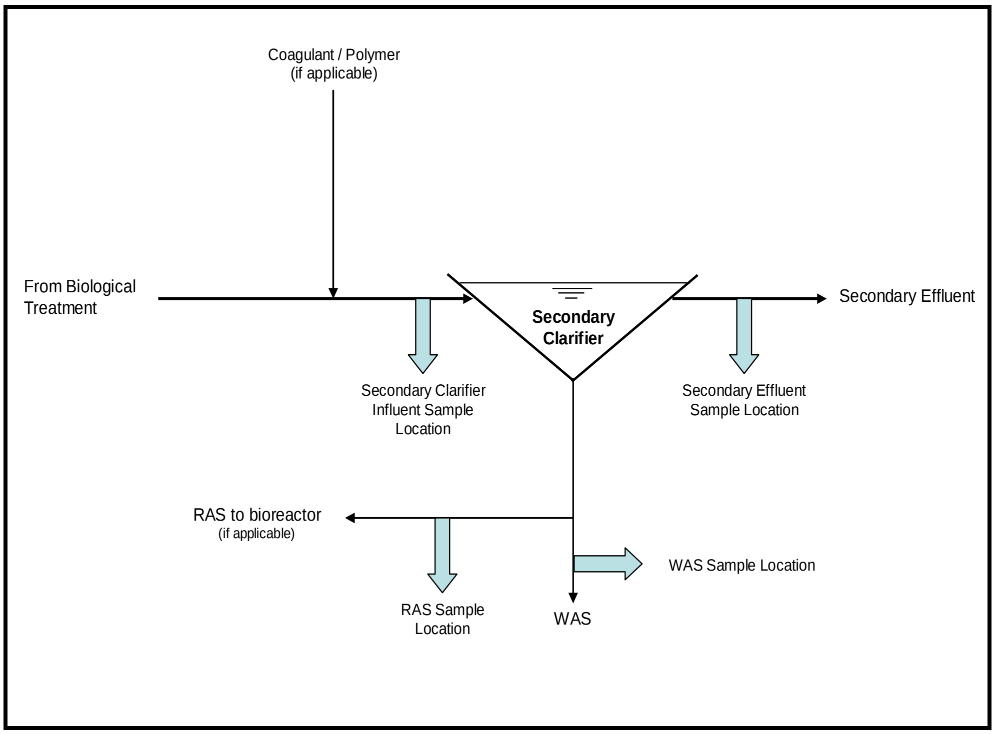

Figure 14-1 – Secondary Clarification – Process Schematic and Sampling Locations

Typically, secondary clarifier performance is evaluated based on the achieved secondary effluent $CBOD_5$, TSS and TP concentrations, and the concentration of return activated sludge withdrawn from the clarifiers. Because secondary effluent parameter concentrations are also affected by the performance of the upstream biological treatment process (Chapter 12), these impacts should be taken into consideration when using secondary effluent quality to assess secondary clarifier performance. Table 14-2 presents typical process performance for the secondary clarifiers for various operating conditions.

Table 14-2 – Secondary Clarification – Typical Process Performance

Operating Condition	Typical Secondary Effluent Concentrations (mg/L) (1)			Typical RAS TSS Concentration (mg/L) (2)
	$CBOD_5$	TSS	TP	
Suspended growth / hybrid biological treatment process without chemical TP removal.	15	15	3.5	4,000 to 12,000
Suspended growth / hybrid biological treatment process with chemical TP removal.	15	15	< 1.0	4,000 to 12,000
Fixed-film biological treatment process without chemical TP removal.	15	20	3.5	n/a
Fixed-film biological treatment process with chemical TP removal.	15	20	< 1.0	n/a

Notes:
n/a – not applicable
1. MOE (2008).
2. Metcalf & Eddy (2003).

It should be noted that the secondary clarifier performance parameters, as presented in Table 14-2, are typical values for wide range of treatment facilities with varying bioreactor and clarifier configurations. Some treatment plants are capable of consistently achieving secondary effluent with TSS and $CBOD_5$ concentrations of less than 10 mg/L and TP concentrations of less than 0.5 mg/L. A clarifier’s ideal performance is based on both the characteristics of the secondary clarifier influent and site-specific tank and channel configuration and hydraulics. Approaches that can be used to determine ideal clarifier performance are presented in Section 14.4.4.

14.1.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the secondary clarification process are presented in Table 14-3.

Table 14-3 – Secondary Clarification – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Uneven flow distribution among clarifiers.	• Some clarifiers are overloaded, potentially resulting in poor effluent quality due to limited settling • Other clarifiers are underloaded, potentially resulting in stagnant, septic conditions, reducing effluent quality due to rising septic sludge and/or causing odours • Uneven rate of effluent flow between secondary clarifiers visible in effluent launders	• Uneven weir levels among clarifiers • Uneven weir lengths among clarifiers • Uneven weir levels within a clarifier (Section 14.2.2) • Poor hydraulics of upstream flow control devices
Hydraulic short-circuiting within clarifiers.	• Reduced secondary clarifier effluent quality • Stagnant, septic regions and regions of high flow and poor settling within clarifier • Erratic clarifier performance	• Poor design of inlet structures and in-clarifier baffling (Section 14.2.2) • Uneven weir levels within a clarifier (Section 14.2.2) • Density currents due to temperature gradients, and wind-driven circulation cells (Section 14.2)
Long sludge retention time within clarifier.	• Development of septic, rising sludge, reducing secondary effluent quality and potentially causing odours • Rising sludge due to denitrification within the sludge blanket, resulting in the formation of nitrogen gas bubbles (occurs in nitrifying plants) • Deep sludge blanket, resulting in decreased effluent quality due to solids carryover, especially during high flow events	• Poor control of RAS pumping (Section 14.3) • Oversized clarifier

Table 14-3 – Secondary Clarification – Symptoms and Causes of Common Problems (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Short sludge retention time.	• Low RAS TSS concentrations and high operating solids loading rate (SLR), potentially resulting in solids carryover and reduced effluent quality • Little to no sludge blanket and/or ill-defined sludge blanket interface	• Poor control of RAS pumping (Section14.3)
Poor clarifier performance due to poor biomass settling characteristics.	• Bulking sludge or pin floc, due to the conditions of operation and performance of upstream bioreactors • High mixed liquor SVI for activated sludge plants • Low RAS TSS concentrations, rising sludge, and/or poor effluent quality	• Operation of upstream bioreactors resulting in poorly settling biological floc, potentially through poor SRT control (Chapter 12)
Poor clarifier performance not attributable to problems identified above.	• Effluent quality poorer than typical values • Rising sludge resulting in deterioration in effluent quality	• Characteristics of secondary clarifier influent not conducive to good settling performance (Chapter 12) • Clarifiers hydraulically overloaded from operating at flows exceeding their design values • Clarifiers hydraulically underloaded from operating at flows significantly below their design values • Scum carry-over due to poor performance of scum collection system and/or improper scum baffle installation • Foam / scum carry-over due to non-biodegradable surfactants in the plant influent, resulting in overloading of the scum collection system and potential degradation of effluent quality

Table 14-3 – Secondary Clarification – Symptoms and Causes of Common Problems (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
		• Algae growth within the clarifier, especially in nitrifying plants • Cold wastewater temperatures leading to reduced settling rates

14.2 SHORT-CIRCUITING

14.2.1 Causes and Impacts on Performance

In an ideal clarifier, all influent would have the same hydraulic residence time within the clarifier, equal to the ratio of the volume of the clarifier to the influent flow rate. In practice, however, clarifiers are subject to non-ideal flow conditions.

Short-circuiting occurs when a portion of flow reaches the outlet of the clarifier prior to the bulk of the flow that entered the clarifier at the same time. Short-circuiting can lead to deterioration in clarifier performance due to a reduction in effective clarifier volume available for settling, and solids carryover due to localized velocity gradients.

The causes of short-circuiting in secondary clarifiers are similar to those in primary clarifiers, namely density currents, wind-driven circulation cells, and poor clarifier design. A summary of the possible causes of short-circuiting within a clarifier was presented in Section 11.4.1.

In secondary clarifiers, density currents can not only form due to temperature differences between the influent and contents of the clarifier, but also due to the settling action of the activated sludge floc in suspended growth systems. These density currents can cause jet-like flow patterns within the clarifier.

The extent of short-circuiting within a clarifier can be evaluated by conducting tracer testing (Section 14.4.1). The results of multiple tracer tests can be used to identify if the clarifier hydraulics are consistent (similar tracer response curves between tests), or erratic (dissimilar tracer response curves between tests).

If consistently poor clarifier hydraulic performance is observed, then there is likely a design limitation affecting the clarifier's performance. Installation of baffles, or modification of inlet structures may be able to improve clarifier performance (Section 14.2.2).

If multiple tracer tests identify erratic hydraulics within the clarifier, it is more likely that clarifier performance is being impacted by density currents or wind-driven circulation cells.

14.2.2 Inlet Structures, Outlet Structures and Baffling

The performance impacts of inlet structures, outlet structures, and baffles in secondary clarifiers are similar to those in primary clarifiers. These were discussed in detail in Section 11.4.2.

Two types of baffles commonly used in centre-feed circular secondary clarifiers to alleviate the impacts of density currents are McKinney and Stamford baffles. These are depicted graphically in Figure 14-2.

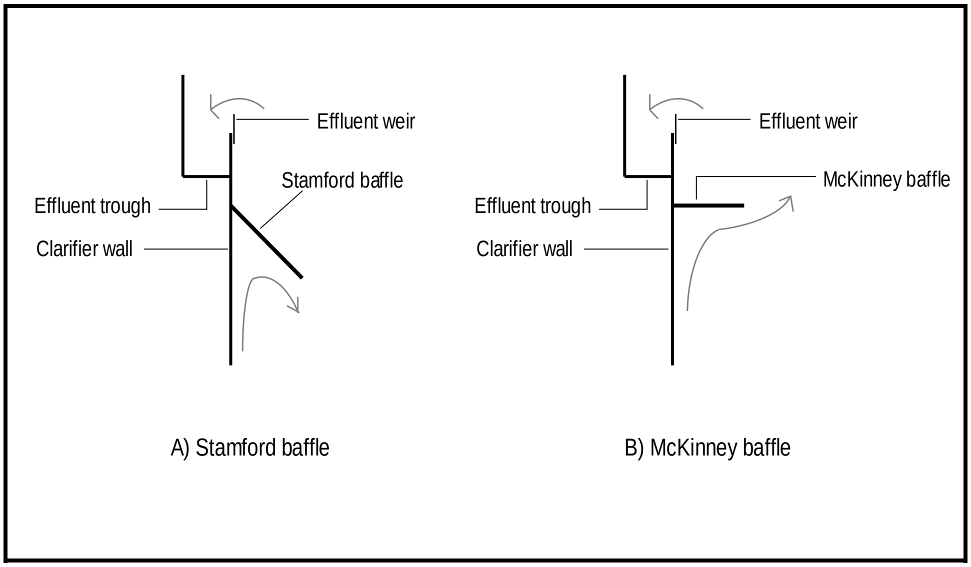

Figure 14-2 – Stamford and McKinney Baffles

Two types of effluent weirs are commonly used in secondary clarifiers, namely straight-edged and V-notched weirs. These are depicted graphically in Figure 14-3. As noted in Chapter 11, replacement of straight edged weirs with V-notched weirs may improve performance as imperfectly levelled V-notched weirs are not as susceptible to non-uniform effluent flow as imperfectly levelled straight edged weirs (WEF/ASCE, 1998).

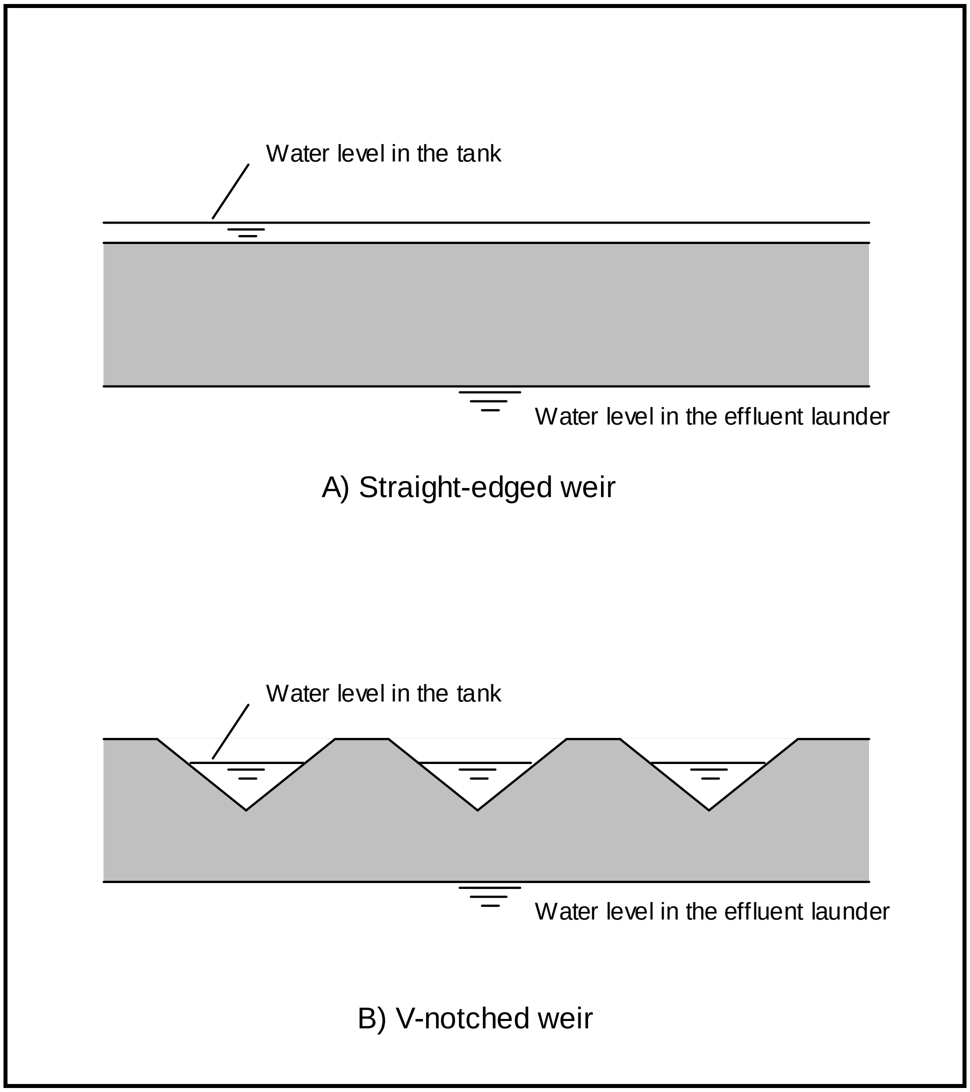

Figure 14-3 – Front View of Common Effluent Weir Types

(Adapted in part from von Sperling, 2007)

14.3 RETURN SLUDGE PUMPING

14.3.1 Impact on Performance

The purpose of RAS pumping in a suspended growth or hybrid treatment process is to maintain adequate MLSS concentrations in the bioreactor(s), while controlling the sludge blanket level in the secondary clarifier(s) to avoid solids washout from the secondary clarifiers. Improper RAS pumping control could potentially result in several negative process impacts:

- If the rate of RAS pumping is inadequate, this can result in:
  - Insufficient MLSS concentrations in the bioreactor(s), resulting in a deterioration of biological treatment; and

    - High sludge blanket levels, resulting in the potential carryover of solids, especially during high flow conditions, or denitrification in the sludge blanket resulting in rising sludge, leading to deterioration in secondary effluent quality.

- If the rate of RAS pumping is excessive, this can result in:

    - Low RAS TSS concentrations and a limited or ill-defined sludge blanket; and

    - High solids loading rates on the secondary clarifiers, resulting in the degradation of effluent quality due to solids carryover.

Optimizing RAS pumping will prevent a deterioration of secondary clarifier performance. The advantages of optimizing RAS pumping include:

- Development of a healthy sludge blanket, that allows for adequate thickening of activated sludge prior to pumping;

- Increased process robustness in terms of more consistent effluent quality at both low and high flow conditions; and

- Appropriate hydraulic loading of the bioreactors and secondary clarifiers.

It should be noted that the optimum RAS pumping rate varies with time and operational conditions at an STP. Various factors can influence the optimal RAS pumping requirements for a specific facility, including:

- Changes in bioreactor operating MLSS concentrations;

- Changes in the settleability of the activated sludge; and

- Changes in the influent flow rate.

As such, operational data should be evaluated on a continuous basis to modify RAS pumping rates to ensure ongoing effective performance of the secondary clarifiers.

14.3.2 Optimizing Return Sludge Pumping

Several strategies have been developed to optimize RAS pumping (WEF, 1997), including:

- Fixed RAS flow control;

- Constant ratio RAS flow control;

- Blanket level control;

- Control based on sludge settling characteristics; and

- Control based on solids flux theory.

With fixed RAS flow control, the RAS pumping rate is set to a constant flow setpoint. If the setpoint is properly selected, this method can minimize suspended solids in the effluent and avoid denitrification within the sludge blanket. However, this method may not maximize sludge thickening, and will result in bioreactor MLSS concentrations, RAS TSS concentrations, and sludge blanket levels in the clarifiers varying with time and operating condition.

Constant ratio (or variable) RAS flow control is based on maintaining a constant ratio of RAS flow to influent flow to secondary treatment. This would require the use of a feedback control loop which compares the measured flow ratio to the setpoint. RAS pump speed and/or flow control valves are modulated to maintain the setpoint. If the flow ratio setpoint is properly selected, this strategy can provide better control than the fixed RAS flow control by maintaining more constant MLSS concentrations, RAS TSS concentrations, and sludge blanket levels in the clarifiers.

Sludge blanket level control relies on maintaining a relatively constant sludge blanket depth in the clarifiers. The sludge blanket level can be measured manually, or by on-line instrumentation. The RAS flow rate is adjusted to maintain the sludge blanket level setpoint. This type of control system is well suited to optimizing sludge thickening within the clarifiers. However, waste activated sludge control (see SRT control, Section 12.3) strategies should be used in conjunction with this type of control strategy, as both control strategies will have an impact on blanket levels.

Sludge settling characteristics can be used to determine the setpoints for both the fixed and constant ratio RAS flow control strategies. A mathematical relationship is used to determine the RAS to influent flow ratio setpoint based on the 30-minute settled sludge volume, as determined as part of SVI testing (Section 12.2.4). The mathematical relationship is:

$$Q_r = \frac{100}{\frac{100}{[MLSS] \times SVI} - 1}$$

where Q_r is the return activated sludge flow rate as a percent of influent flow rate (%);

$[MLSS]$ is the mixed liquor suspended solids concentration (%); and

SVI is the sludge volume index (mL/g).

The control of RAS flow based on solids flux theory uses the state point concept to manipulate the RAS flow to ensure that the secondary clarifier is not overloaded with respect to hydraulic load or thickening capability. The development of settling flux curves and the determination of the state point is discussed in Section 14.4.2.

Details regarding the manipulation of RAS rates based on the operating state point and settling flux curves is described elsewhere (WEF, 1997).

14.3.3 Other Optimization Methods

Proper flocculation of the secondary clarifier influent mixed liquor reduces DSS concentrations, which can help to improve secondary clarifier effluent quality. Modifications to combination flocculation/inlet structures and/or upstream channels and piping can improve the flocculation of the secondary clarifier influent and reduce flocculation non-idealities (Section 14.4.4).

The addition of polymers to secondary clarifier influent has been shown to result in the formation of bridges between floc particles, resulting in larger, stronger, and more readily settleable flocs. The addition of polymer has been shown to allow clarifiers to operate at SOR values approximately 50 percent higher than without polymer, with no associated increase in effluent TSS concentrations (Patoczka *et al.*, 1998). Because the addition of polymer can result in an almost instantaneous improvement in clarifier performance, polymer addition can be used as a strategy to treat wet weather flows. The addition of polymer may, however, have an impact on the density and viscosity of the settled sludge, potentially requiring modifications to the sludge withdrawal piping, pumping and/or sludge collection mechanisms.

14.4 FIELD INVESTIGATIONS

The following sections outline field investigations that can be used to identify process limitations, or to evaluate the impact of implementing optimization measures on secondary clarifier performance.

14.4.1 Tracer Testing

Tracer testing can be used to identify hydraulic short circuiting within a clarifier and/or to quantify uneven flow distribution among secondary clarifiers. In sewage treatment processes, the most commonly used tracers are fluorescein, rhodamine WT and Pontacyle Brilliant Pink B as they can be detected at low concentrations using a fluorometer (Metcalf & Eddy, 2003).

Crosby Dye Testing

Dye testing based on the method presented by Crosby (1987) consists of two separate tests, namely Dispersion Testing and Flow Pattern / Solids Distribution Testing. Both components of the Crosby Dye Testing procedure are outlined below.

Dispersion Testing

Dispersion testing can be used to evaluate the hydraulic non-ideality of flow through a clarifier. Dispersion testing usually consists of the following steps:

- Select effluent sampling locations. In the case of rectangular clarifiers with a single, straight effluent weir, one effluent sampling location near the discharge of the effluent launder can be selected. For circular clarifiers, or rectangular clarifiers with various longitudinal effluent weirs, several sample locations are needed. Sampling locations should be selected based on observation of flow patterns in the clarifier, such as areas of low flow, high flow, or those with recurring solids upflow;

- Collect background effluent samples prior to releasing dye;

- Add a slug (pulse input) of tracer upstream of, or at the inlet to, the clarifier to be tested. The time at which the slug load enters the clarifier is noted as “time zero”. The tracer should be added in an area that will provide sufficient mixing; in some instances an auxiliary mixer may be required. The amount of tracer required will vary depending on the flow rate and composition of the clarifier influent, and the type of tracer used;

- Collect a series of grab samples from the effluent sampling locations and analyze them for tracer concentration, beginning at “time zero”. Samples should be collected at intervals of 5 minutes for a minimum of 30 minutes. Subsequent sample intervals can be reduced to 10 minutes for the first hour, 15 minutes for the second hour, and 30 minutes for the third hour. The last sample should be collected no less than two theoretical hydraulic detention times after “time zero”;

- In some instances, online instruments can also be used to measure tracer concentration in the effluent over time; however, several grab samples should also be collected and analyzed in order to verify the results from the online instrumentation; and

- The effluent tracer concentration data are used to develop tracer response curves. An example of a tracer response curve for a dispersion test, along with the ideal response curve, is presented in Figure 14-4.

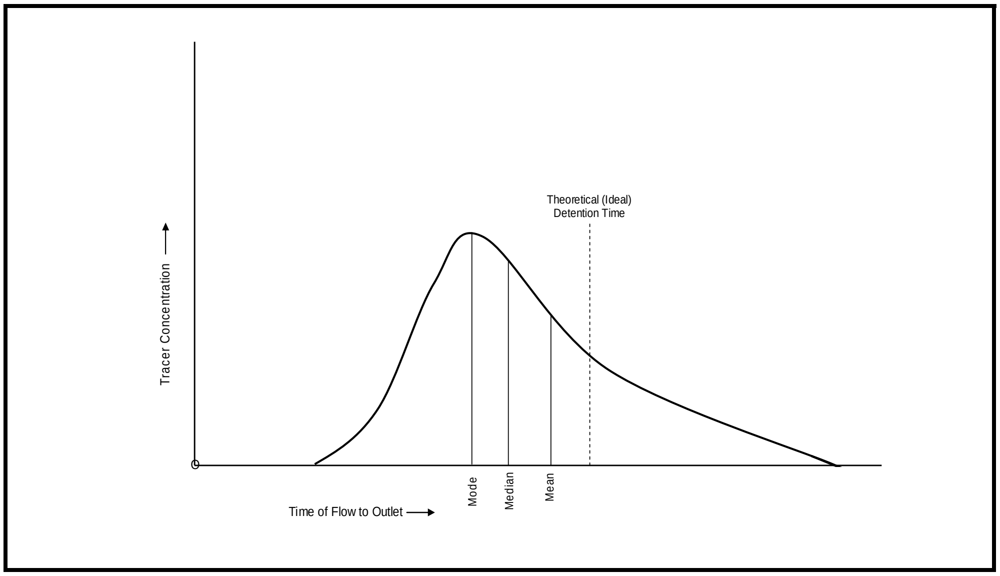

Figure 14-4 – Example Tracer Response Curve

(Adapted from Metcalf & Eddy, 2003)

Variations between the observed response and ideal curve are used to evaluate the hydraulic efficiency of the clarifier. Methods available to evaluate the results of the dispersion testing are outlined elsewhere (Metcalf & Eddy, 2003; Daigger and Buttz, 1998).

Flow Pattern / Solids Distribution Testing

Flow pattern / solids distribution testing can be used to generate graphical representations of the flow pattern along the cross section of a clarifier, and to quantify the distribution of solids within the clarifier. Flow pattern / solids distribution testing usually consists of the following steps:

- Continuously feed dye into the clarifier influent at a point that provides good mixing of the dye into the influent stream;
- Sample stations are selected along the length of a rectangular clarifier or along the radius of a circular clarifier. The stations are located such that they are evenly distributed from the influent feed well to the effluent weirs;
- At each sample station, samples are collected simultaneously from five different depths using sample pumps. The five depths are selected such that they are evenly distributed from the clarifier surface and the top of the sludge blanket; and
- The five depth samples are collected at each sample station in rapid succession, so that a 'snapshot' of the clarifier performance characterstics can be developed. Sample collection is then repeated to produce additional 'snapshots' at various points during the test.

Dye and TSS concentrations in each sample collected are used to develop graphic representations of dye and solids profiles for each 'snapshot'. A typical dye pattern snapshot is shown in Figure 14-5. The blue dots represent sampling locations, while the solid lines represent interpolated lines of constant concentration (isolines).

The dye and solids profiles gathered throughout the testing are used to identify dead zones, jetting problems, and density currents. Examples of the results of flow pattern / solids distribution testing are outlined elsewhere (Daigger and Buttz, 1998).

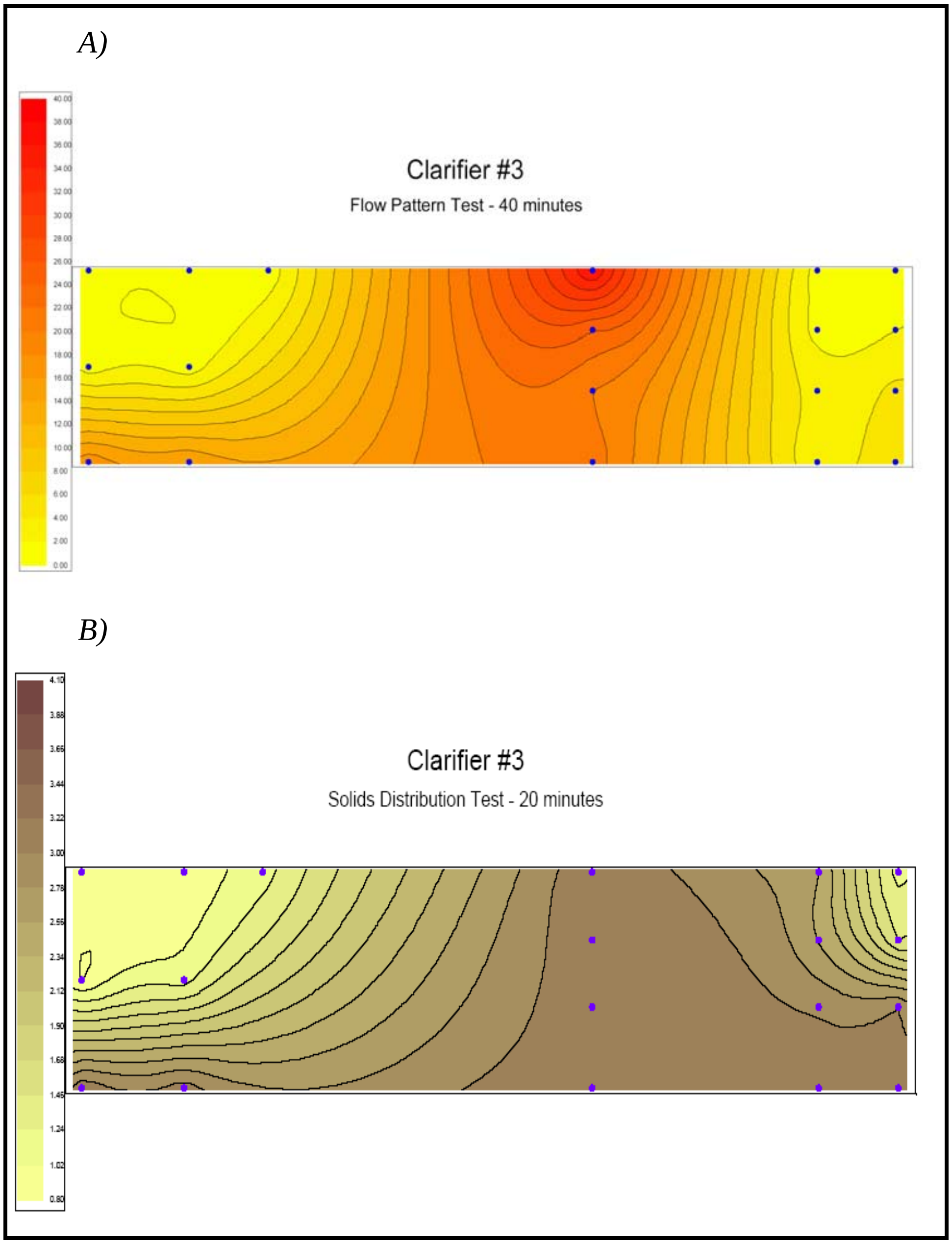

Figure 14-5 – Example A) Flow Pattern and B) Solids Distribution Profiles

Tracer Testing to Evaluate Flow Distribution between Clarifiers

The results of dispersion testing can be used to evaluate the flow distribution between clarifiers by conducting simultaneous tracer testing on clarifiers operating in parallel.

Assuming identical clarifiers, uniform mixing of dye upstream of each tank, and equivalent “time zero” for each clarifier, the median tracer retention time observed in a clarifier is inversely proportional to flow rate through that clarifier.

14.4.2 State Point Analysis

State point analysis can be used to determine if a secondary clarifier is overloaded, critically loaded, or underloaded with respect to both its clarification and thickening capacities.

Development of a state point analysis plot requires the following:

- A settling solids flux (mass flow of solids per unit cross sectional area) curve, which shows graphically the relationship between solids flux and suspended solids concentration;

- The surface overflow rate operating line, which intersects the origin and has a slope equivalent to the ratio of secondary influent flow rate to clarifier surface area; and

- The underflow rate operating line, which intersects the y-axis at the operating clarifier solids loading rate and has a negative slope equivalent to the ratio of RAS flow rate to clarifier surface area.

The settling solids flux curve is developed based on the results of settling tests conducted on the mixed liquor. More information regarding the development of settling solids flux curves can be found in WERF (2009) and Metcalf & Eddy (2003).

Once plotted, the intersection point of the surface overflow rate and underflow rate operating lines represents the "state point". The location of the state point and underflow rate operating line, in relation to the settling solids flux curve, can then be used to identify if the clarifier is underloaded, critically loaded, or overloaded with respect to both clarification and thickening capacities. Examples of several state point analysis plots are presented in Figure 14-6.

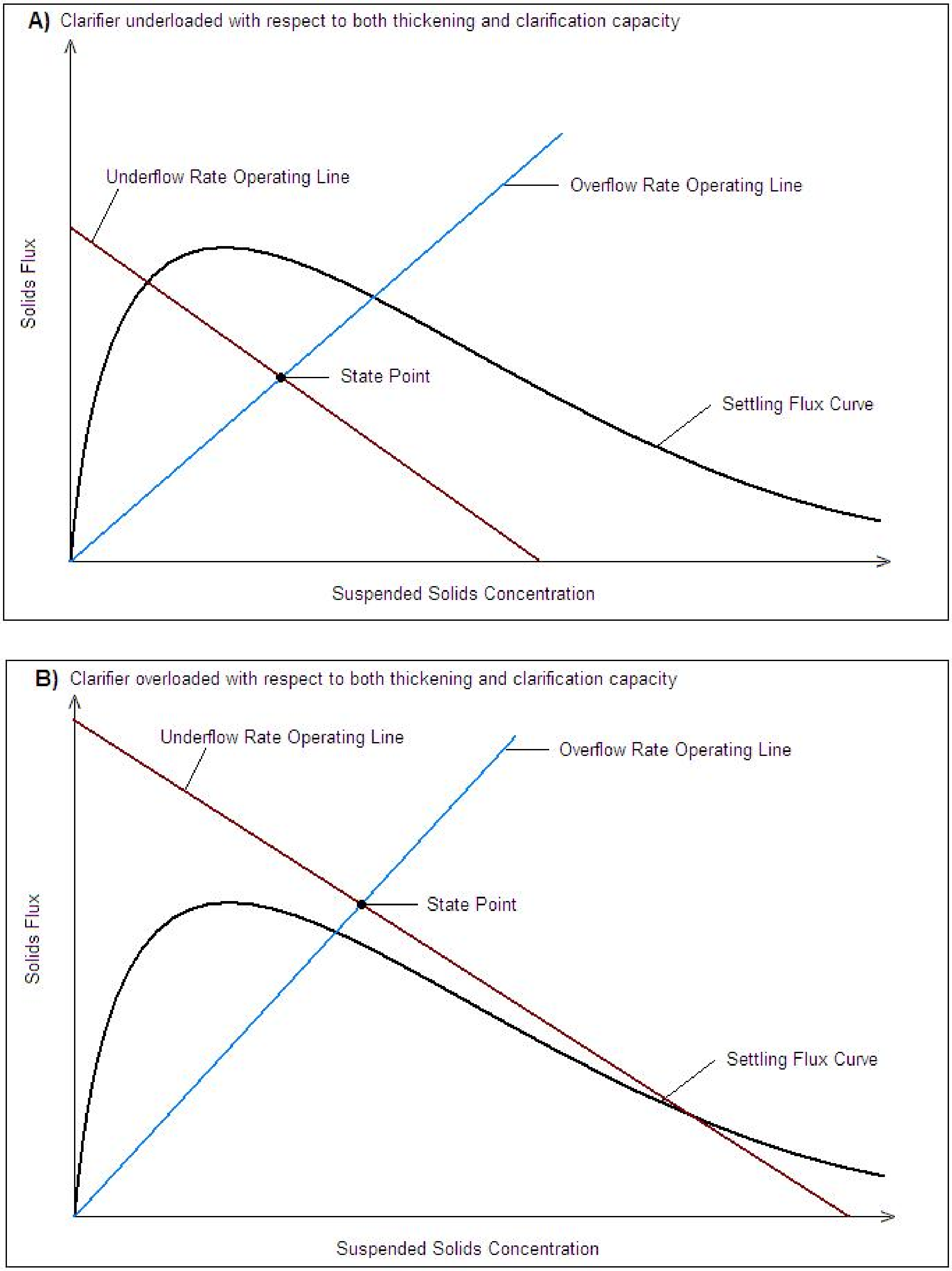

Figure 14-6 – Example State Point Analysis Plots

In the case of an overloaded clarifier, the results presented in the state point analysis plot can be used to make operational changes to shift the operating point of a clarifier to an underloaded state.

More information regarding the development and interpretation of the results of state point analysis can be found in WERF (2009) and Metcalf & Eddy (2003).

14.4.3 Stress Testing

Stress testing can be used to quantify the capacity, both in terms of clarification and thickening, of a secondary clarifier.

Stress testing involves incrementally increasing the flow rate to the test clarifier(s) by, for example, taking other clarifiers off-line or pumping the contents of an off-line clarifier / tank into the aeration tank influent channel. The influent or effluent flow rate from the test clarifier(s) is recorded throughout the duration of the test, along with the TSS and MLSS concentrations of the secondary effluent and mixed liquor influent, respectively. The SVI of the influent mixed liquor should also be recorded, as the observed capacity of the test clarifier(s) is a function of the settling properties of the activated sludge solids.

Throughout the test, influent flow rate to the test clarifier(s) is increased incrementally until failure of the clarifier is observed, either by exceeding final effluent TSS concentration targets or by a loss of thickening as observed by an increasing sludge blanket depth.

The clarification capacity of a secondary clarifier is generally assessed based on the ability of the clarifier to meet effluent TSS performance standards. The thickening capacity is generally assessed based on the ability of the clarifier to maintain a steady sludge blanket level, thereby avoiding washout.

More detailed information regarding typical stress testing protocols can be found elsewhere (Daigger and Buttz, 1998; Ekama *et al.*, 1997).

14.4.4 Determining Ideal Clarifier Performance

The characteristics of the secondary clarifier influent can be evaluated to determine the ideal performance expected from a secondary clarifier.

Because a secondary clarifier relies on settling to remove suspended solids and activated sludge floc, only the settleable fraction of these constituents can be removed during the secondary clarification process.

Dispersed suspended solids (DSS) and flocculated suspended solids (FSS) testing can be used to determine hydraulic and flocculation non-idealities which impact clarifier performance in terms of effluent suspended solids (ESS) concentrations. Testing procedures and data analysis techniques are outlined in Ekama *et al.* (1997).

The difference between DSS and ESS concentrations can provide information regarding the potential impact of hydraulic non-idealities within the clarifier and/or sludge blanket management issues on clarifier performance. The difference between FSS and DSS concentrations can provide information regarding the potential impact of flocculation non-idealities on clarifier performance.

For secondary clarifiers in activated sludge processes, DSS testing can be performed on samples collected at both the bioreactor effluent and the secondary clarifier influent to determine if floc breakup is occurring in channels, pipes, or other areas between the bioreactors and the clarifiers. In such cases, optimization of flocculation may be possible through the modification of these areas to promote flocculation and avoid turbulent zones causing floc breakup (Section 14.3.3).

Clarifier models, using CFD, can also be used to predict clarifier performance based on the hydrodynamic conditions within the clarifier (Chapter 6).

For information regarding the details of required testing and data analysis, reference should be made to Ekama *et al.* (1997).

14.5 CASE HISTORIES

14.5.1 Woodward Avenue WWTP, Hamilton, Ontario – Optimization of Secondary Clarification

The following case study is based on information presented in CH2M Hill and Hydromantis Inc. (2004).

As part of an overall WWTP optimization project, the condition and performance of the existing secondary clarifiers at the Woodward Avenue WWTP were evaluated, with the objective of identifying operational or design modifications that would lead to improved clarifier performance.

At the time of the study, the South Plant clarifiers were scheduled for major structural upgrades, and as such were not evaluated as part of the optimization study. Two North Plant clarifiers were tested simultaneously: Clarifier No. 8 – a circular clarifier equipped with baffles (McKinney effluent and mid-radius ring baffles), and Clarifier No. 4 – a circular unbaffled clarifier.

Two separate stress tests were conducted. Test No. 1 (“Low MLSS”) was conducted at an influent MLSS concentration of approximately 1,940 mg/L, while Test No. 2 (“High MLSS”) was conducted at an influent MLSS concentration of approximately 2,560 mg/L.

Based on the results of the stress testing, Clarifier No. 8 (baffled) performed better than Clarifier No. 4 (unbaffled) during Test No. 1 (“Low MLSS”); however, Clarifier No. 4 (unbaffled) performed better than Clarifier No. 8 (baffled) during Test No. 2 (“High MLSS”). It was noted that the sludge blanket level in the baffled clarifier was consistently higher than that in the unbaffled clarfier during both tests.

Based on the results of bench scale testing and full scale stress testing, it was determined that the influent mixed liquor was poorly flocculated, and that improvements to the inlet works, such as the installation of flocculation baffles at the clarifier inlet to improve flocculation, would enhance effluent quality. In addition, the modification of existing baffles, where applicable, and the installation of new baffles in unbaffled clarifiers could also improve effluent quality.

14.5.2 Various Wastewater Treatment Facilities, United States – Optimization of Secondary Clarifiers

The following case study is based on information presented in Parker *et al*. (2000).

City of Tolleson, Arizona

Denitrification, leading to rising sludge and high ESS concentrations, was being experienced in two 34 m diameter flocculator clarifiers.

The following corrective actions were taken to optimize clarifier performance: the RAS flow rate was increased to reduce solids retention time in the clarifier, and DO level in the mixed liquor entering the secondary clarifiers was increased to prevent the development of anoxic conditions within the clarifier.

While these operational changes improved the performance of the clarifiers, rising sludge due to denitrification was still observed. As a final optimization step, one clarifier was taken offline, and the speed of the sludge collection mechanism in the online clarifier was increased. This reduced the solids retention time in the online secondary clarifier, eliminating the rising sludge and reducing the ESS concentration.

City of Atlanta, Georgia

High sludge blanket levels were an ongoing operational concern, even at low solids loading rate (SLR) conditions. State point analysis was used to determine that the clarifiers were operating at an underloaded state, both in terms of thickening and clarification capacities.

Based on the analysis, it was determined that the recorded RAS flow rate was potentially suspect. The accuracy of the RAS flow meters was assessed, and they were found to be accurate.

An assessment of the vacuum sludge removal system identified leaking seals that were allowing clear supernatant into the RAS lines, reducing the solids removal rate from the clarifiers. The defective seals were replaced, which led to a significant reduction in the operating blanket levels.

Greeley Water Pollution Control Facility (WPCF), Colorado

DSS/FSS testing was conducted to determine opportunities for optimizing existing secondary clarifiers at the Greeley WPCF. Based on testing results, it was determined that clarifier performance was being limited by the poorly flocculated state of the clarifier influent, and poor flocculation within the clarifiers themselves.

Modifications were made to the clarifiers by converting the inlet structure to a flocculator, and providing an area for flocculation outside the inlet well by installing a metal skirt baffle around the outlet ports.

The modifications improved flocculation within the clarifier, and significantly improved ESS concentrations.

Central Marin Sanitation Agency, California

High ESS concentrations were observed from four circular, centre-feed clarifiers during peak flow events. DSS profiling indicated that some flocculation was occurring within the clarifiers, but that hydraulic inefficiencies were resulting in floc carry-over, thus causing the elevated ESS concentrations.

A hydraulic model was developed and calibrated. The calibrated model was then validated using the results of dispersion dye testing.

The model identified the hydraulic limitations based on the geometry of the test clarifiers. The model was then used to predict the impact of various modifications, including the installation of baffles and modifying the configuration of the influent feed well, on clarifier performance. The results of the modelling were used to determine the most cost effective modifications to improve clarifier performance.

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CHAPTER 15

TERTIARY TREATMENT PROCESSES

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TERTIARY TREATMENT PROCESSES

15.1 GRANULAR MEDIA FILTERS

15.1.1 Purpose and Types of Filters

Granular media filtration is an advanced treatment process that removes TSS and particulate phosphorus to a higher degree than secondary treatment alone. The solids in the secondary effluent (filter influent) are removed by a variety of mechanisms as the influent passes through the filter. Generally, the particulates are retained by the filter grains or previously deposited particulates involving a number of possible removal mechanisms such as straining, interception, impaction, sedimentation, flocculation, and adsorption (Metcalf & Eddy, 2003).

Typical configurations include: single-, dual-, mixed-media; shallow-, conventional-, deep-bed; upflow, downflow or biflow; pressure or gravity filters; and continuous or semi-continuous backwash filters.

The most common types of systems in wastewater applications are described in Table 15-1.

Table 15-1 – Common Types of Granular Media Filters

(Adapted from Metcalf & Eddy, 2003)

Type	Description
Conventional Downflow Filters	• Filter media can consist of single-, dual- or multi-media • Backwashing is primarily achieved by water wash with surface water scour or by water wash with air scour
Deep-Bed Downflow Filters	• Similar to conventional downflow filters with the exception of the filter bed depth and size of filtering media • More solids can be stored within the filter and consequently a longer run time is achieved from the filters greater depth and larger sized media than conventional filters • Typically air scour and water is used during the backwash operation
Deep-Bed Upflow Continuous Backwash Filters	• Wastewater flows upwards through the downward moving sand • Filtrate leaves the sand bed, overflows a weir and is discharged from the filter • Sand particles with the retained solids are suctioned downward while impurities are scoured off

Table 15-1 – Common Types of Granular Media Filters (continued)

(Adapted from Metcalf & Eddy, 2003)

Type	Description
Pulsed Bed Filters	• Proprietary downflow gravity filter with a filter media consisting of unstratified shallow layer of fine sand • Air pulses disrupt the sand surface and allow penetration of TSS into the bed • Backwash cycle is initiated when terminal headloss is reached
Traveling Bridge Filters	• Proprietary continuous downflow, automatic backwash, low-head, medium depth filter • Filter bed is divided horizontally into independent filter cells which contain media • Each cell is backwashed individually by an overhead, traveling bridge assembly • Wastewater is continuously filtered through the cells that are not being backwashed

15.1.2 Evaluating Process Performance

Table 15-2 presents a typical granular media filter monitoring program, in terms of sampling locations and analyses that would be used to evaluate the performance of the process.

Table 15-2 – Granular Media Filters – Minimum Recommended Process Monitoring

Location	Types of Sample/ Measurement	Parameters/ Analyses	Comments
Tertiary Influent (secondary effluent)	Grab and/or Continuous On-line	• Turbidity • TSS • TP • $CBOD_5$	Continuous turbidity analysis of the filter effluent can be conducted using an online turbidity meter. TSS, TP, soluble P and $CBOD_5$ samples are submitted for laboratory analysis.

Table 15-2 – Granular Media Filters – Minimum Recommended Process Monitoring (continued)

Location	Types of Sample/ Measurement	Parameters/ Analyses	Comments
Tertiary Effluent from Each Filter	Grab and/or Continuous On-line	• Turbidity • TSS • TP • ortho-phosphorus or soluble P • $CBOD_5$	See comments for tertiary influent. Filters only remove particulate phosphorus. Soluble phosphorus will remain constant in the influent and effluent. To determine filter removal efficiency, utilize the measured effluent soluble phosphorus concentration, and the influent and effluent TP concentrations to determine particulate phosphorus removal across the filter. Tertiary effluent samples should be collected on the same day as tertiary influent samples so that removal efficiencies across each filter can be calculated.

In addition to the recommended sample locations and analyses presented in Table 15-2, it is recommended that the following be monitored:

- Hydraulic loading rate;
- Floc strength measured by dispersed suspended solids/flocculated suspended solids (DSS/FSS) analysis;
- Quantity (volume and dosage) of chemicals applied to upstream processes; and
- Backwash rate, frequency and duration.

Filtration rate affects the performance of the filters. Observations indicate that filtration rates in the range of 80 to 320 L/m^2·min (19.2 m/hr) will not affect the effluent quality when filtering settled activated sludge effluents (Metcalf & Eddy, 2003). Gravity filters typically operate with filtration rates ranging from 5 to 15 m/h and terminal headloss ranging from 2.4 to 3 m (WEF/ASCE, 1998). Pressure filters typically operate with higher filtration rates and headloss than gravity filters. Pressure filters can operate with filtration rates of 20 m/h and terminal head losses up to 9 m (WEF/ASCE, 1998).

The optimum rate of filtration depends on the strength of the floc and the size of the filtering media.

If floc strength is weak, the floc particles can shear and carry through the filters resulting in a poor quality filter effluent (Reddy and Pagilla, 2009). Upstream chemical coagulant addition can be used to increase floc strength and improve filter performance and/or capacity.

Figure 15-1 presents a process schematic of typical granular media filtration process, along with the identification of recommended sampling locations.

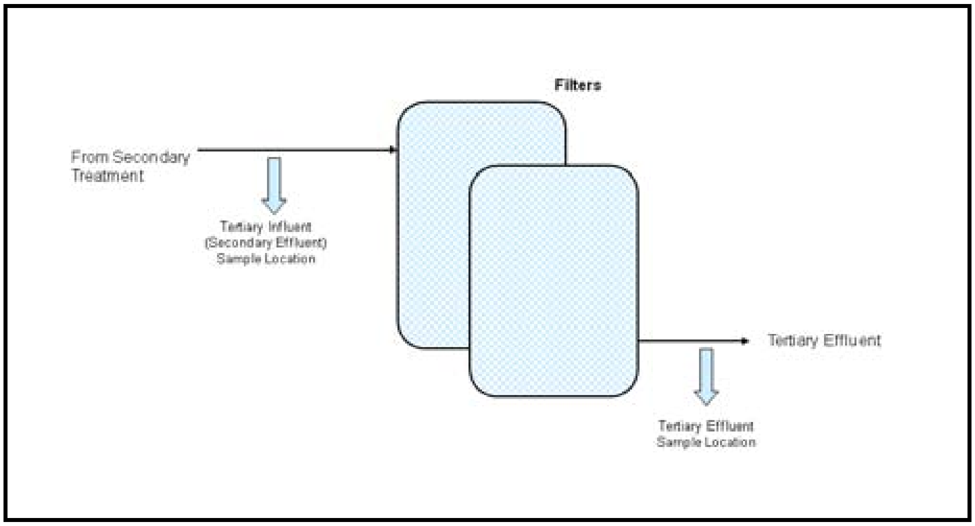

Figure 15-1 - Granular Media Filters - Process Schematic and Sampling Locations

15.1.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the granular media filtration process are shown in Table 15-3.

Table 15-3 – Granular Media Filters – Symptoms and Causes of Common Problems
(Adapted from Metcalf & Eddy, 2003)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low quality effluent.	• High turbidity levels in effluent even though terminal headloss has not been reached	• Turbidity breakthrough caused by improper dosing, addition point of chemicals upstream of tertiary treatment, due to media that has been washed out due to improper backwash rate and/or channeling due to problems with uneven flow distribution during backwash

Table 15-3 – Granular Media Filters – Symptoms and Causes of Common Problems (continued)
(Adapted from Metcalf & Eddy, 2003)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
		• Optimum chemical, dosage and addition point should be determined by jar testing followed by full-scale trials
Biological growth and/or emulsified grease accumulation in filter media.	• Agglomeration of biological floc, dirt and filtering media • Mudball formation. If not removed, the formed mudballs will grow and eventually sink into filter bed reducing effectiveness of filtration and backwashing	• Long filter runs • If applicable, improper dosing and/or injection point of chlorine upstream of tertiary treatment • Inadequate backwashing (i.e. inadequate frequency and/or no air scour or no surface wash step, failure or reduced efficiency of backwash pumps(s))
Development of cracks and contraction of the filter bed.	• Improper cleaning of filter bed causes grains of the filter medium to become coated resulting in the development of cracks, especially on sides of filter • May lead to mudball formation	• Inadequate backwashing (i.e. no auxiliary air and/or no water scouring step present or not optimized, failure or reduced efficiency of backwash pumps(s))
Loss of filter media.	• Grains of filter media become attached to biological floc and are washed away during backwashing • Causes shorter filter runs and reduces effluent quality	• Inadequate backwashing (i.e. no auxiliary air and/or no water scouring step present or not optimized, failure or reduced efficiency of backwash pumps(s)) • Backwash flow rate may be too high and, as a result, media is flushed out • Improper placement of wash water troughs and underdrain systems to prevent loss of media through underdrain system
Gravel mounding.	• Excessive rates of backwashing causing disruption to the various layers of support gravel if used	• Addition of a high-density material such as ilmenite or garnet to gravel support layer

Table 15-3 – Granular Media Filters – Symptoms and Causes of Common Problems (continued)
(Adapted from Metcalf & Eddy, 2003)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Stratification of media.	• Often occurs after backwash in single-media filters • Reduces effectiveness of filter when fine-grained portions of the media concentrate in the upper portion of the filter bed (WEF/ASCE, 1998)	• Changing direction of flow to upflow would achieve fuller use of bed (WEF/ASCE, 1998) • Use of dual media or multi-media beds can alleviate the problem
Frequent backwashing.	• Systems requiring frequent backwashing cycles will use large volumes of water	• Inadequate backwashing (i.e. inadequate frequency and/or no air scour or no surface wash step, failure or reduced efficiency of backwash pumps(s)) • Media size may be too small, causing a high headloss across the filter bed. Can be addressed by adding a media cap of larger size, less dense material, or by replacing the media • Addition of air scouring to the backwashing process will improve the effectiveness of backwashing and reduce the cycle frequency and volume of water
Short filter runs.	• Caused by high hydraulic loading and/or TSS loading	• Flow equalization can minimize flow or load variation

15.1.4 Optimizing Process Performance

The objective of a proper functioning filter is to (WEF/ASCE, 1998):

- Consistently produce effluent of the required quality with varying influent conditions;
- Maintain continuous service under a variety of load conditions; and
- Restore the filter’s capacity through backwashing.

If these objectives are not being consistently met, an optimization study of the filters should be conducted to ensure the filters are functioning properly.

Stress testing can be used to quantify the capacity and performance of the filter under high flow or solids loading conditions. During the test, the flow rate to each filter should be incrementally increased. Effluent quality parameters (i.e. TSS, TP, turbidity) are monitored and compared to the required effluent quality.

During the test, the influent flow rate to each filter is increased incrementally until deterioration in effluent quality is noted. The capacity of the filter is assessed based on the ability of the filter to meet effluent limits for TSS, TP and turbidity at increasing flowrates. Incremental solids loading impacts are used to assess the filter's response to higher solids loading rates due to increased secondary clarifier effluent TSS concentrations.

Chemical addition upstream of the filter can improve the filterability of the influent. Coagulants can be added to the filter influent to increase floc strength or enhance flocculation. Optimized phosphorus removal by alum or iron addition upstream of the tertiary filter (post-precipitation) is discussed in Chapter 16. As described in Chapter 16, jar testing should be done to select the optimum chemical(s), dosage(s) and addition point(s).

The type and size of media affects filter throughput, performance and headloss. Characteristics such as media size, shape, composition, density, hardness and depth can be considered during optimization, although some of these parameters are difficult to change as part of an optimization program. The most common types of media in tertiary filters are anthracite and sand. Problems may arise with the filter due to improper media selection. If the media grain size is too small, headloss during the filter run will increase. If the media grain size is too large, smaller particulate matter in the secondary effluent will not be removed as effectively.

Feed water quality to the tertiary filter is directly related to the performance of the upstream treatment processes. Studies have shown that operational changes upstream can significantly affect the particle size distribution of the filter influent. The cause of filter performance problems often can be identified by an analysis of the distribution of particles by size. By completing a particle count, the response of TSS and turbidity measurements during worsening effluent water quality can be better understood by the operator and possibly remedied (Reddy and Pagilla, 2009). If the particle size distribution analysis shows the tertiary effluent has a high concentration of smaller sized particles which are not removed by the filter, consideration should be given to prefiltration coagulation and flocculation to improve filter performance. An increase in effluent particles can compromise the effectiveness of downstream disinfection processes. Coliform bacteria, a surrogate organism for pathogens, can associate with particles and be shielded from the effects of disinfection resulting in higher coliform counts in the effluent (Reddy and Pagilla, 2009).

Dynamic and continuous turbidity data can be used as part of a filter optimization program to determine the effects of filter hydraulic loading, chemical dosage, backwash cycles, and other factors on filter performance.

Results of long-term testing conclude that average filter effluent turbidity of 2 NTU or less can be achieved if a high quality secondary effluent with turbidity less than 5 to 7 NTU is applied to the filter. To achieve effluent turbidity of 2 NTU or less when influent turbidity is greater than 5 NTU may require the addition of chemicals such as organic polymers. Typically, TSS concentrations range from 10 to 17 mg/L when influent turbidity ranges from 5 to 7 NTU and TSS concentrations range from 2.8 to 3.2 mg/L when effluent turbidity is 2 NTU (Metcalf & Eddy, 2003).

Turbidity and TSS are related through the following equations:

Settled secondary effluent:

TSS, mg/L = (2.0 to 2.4) X (turbidity, NTU) (Metcalf & Eddy, 2003)

Filter effluent:

TSS, mg/L = (1.3 to 1.5) X (turbidity, NTU) (Metcalf & Eddy, 2003)

The actual relationship between TSS and turbidity can vary from plant to plant and should be confirmed by testing if turbidity is used to optimize filter performance.

To optimize and/or improve the removal of turbidity, total suspended solids and phosphorus, two-stage filtration can be utilized. Two filters used in series can produce a high quality effluent. The first filter uses a large-sized sand diameter to increase contact time and minimize clogging. The second filter uses a smaller sand size to remove residual particulates from the first stage filter. Phosphorus levels of less than 0.02 - 0.1 mg/L can be achieved with this process including post-precipitation (Metcalf & Eddy, 2003). Chapter 16 provides more information regarding two-stage filtration for phosphorus removal.

15.2 MEMBRANE FILTERS

15.2.1 Purpose and Types of Membranes Systems

Microfiltration (MF) and ultrafiltration (UF) are membrane filtration processes typically used for advanced treatment of wastewater.

A high quality effluent, referred to as the permeate, is produced by passing the wastewater through a membrane barrier. The permeate passes through the membrane surface while the impermeable components are retained on the feed side creating a reject stream. In the membrane system the particles are removed from the wastewater through surface filtration as the wastewater is passed through the membrane surface and the particles are mechanically sieved out (Metcalf & Eddy, 2003). The rate of the influent passing through the membrane surface is referred to as the “filtrate flux”, and has units of volume/(area·time).

The quality of effluent produced depends on the membrane pore size. Typically microfiltration removes particles sized above 0.1 µm and ultrafiltration removes particles sized above 0.01 µm. Microfiltration in advanced wastewater applications has been used to replace depth filtration to reduce turbidity, remove TSS, remove protozoan cysts and oocysts and helminth ova, and reduce bacteria prior to disinfection (Metcalf & Eddy, 2003). Ultrafiltration produces a higher quality permeate than microfiltration specifically by the removal of some dissolved solids, several cysts, bacteria and viruses and enhances subsequent disinfection practices (MOE, 2008). Similar to granular media filters, membrane filters require backwashing to remove the accumulated solids on the membrane surface and restore their operating capacity. There are two methods of cleaning membranes: membranes by reversing the flow of permeate through the membrane, and by chemical cleaning of the membranes modules to remove attached solids.

Membrane processes that are pressure driven will have high energy requirements. Microfiltration systems operating at 100 kPa (14.5 psi) will have energy consumption of approximately 0.4 kWh/m^3 (Metcalf & Eddy, 2003). Ultrafiltration systems with an operating pressure of 525 kPa (76.1 psi) will consume approximately 3.0 kWh/m^3 (Metcalf & Eddy, 2003).

Membrane filters, like granular media filters, will produce a recycle/reject stream that will need to be returned to the upstream processes for treatment.

Factors which should be considered when designing membrane systems include expected peak flows, minimum temperatures, equalization requirements and/or chemical requirements for post-precipitation.

15.2.2 Evaluating Process Performance

Effluent quality should remain relatively constant with varying influent water quality and operating conditions. Table 15-4 presents a typical membrane process monitoring program, in terms of sampling locations and analyses that would be used to evaluate performance of the process.

Table 15-4 – Membrane Filters – Minimum Recommended Process Monitoring

Location	Types of Sample/ Measurement	Parameters/ Analyses	Comments
Tertiary Influent (secondary effluent)	Grab and/or Continuous On-line	• Turbidity • TSS • TP • $CBOD_5$ • Particle count • Temperature • pH	Continuous monitoring is recommended for temperature, pH, turbidity, particle count. Daily monitoring of TSS and *E. coli* is recommended.

Table 15-4 – Membrane Filters – Minimum Recommended Process Monitoring (continued)

Location	Types of Sample/ Measurement	Parameters/ Analyses	Comments
Tertiary Effluent from Each Filter Bank	Grab and/or Continuous On-line	• Turbidity • TSS • TP • ortho-phosphorus or soluble P • $CBOD_5$ • Particle count	See comments for tertiary influent. Filters only remove particulate phosphorus. Soluble phosphorus will remain constant in the influent and effluent. To determine filter removal efficiency, utilize the measured effluent soluble phosphorus concentration, and the influent and effluent TP concentrations to determine particulate phosphorus removal across the filter.

Figure 15-2 presents a process schematic of typical membrane filtration process, along with the identification of various sampling locations.

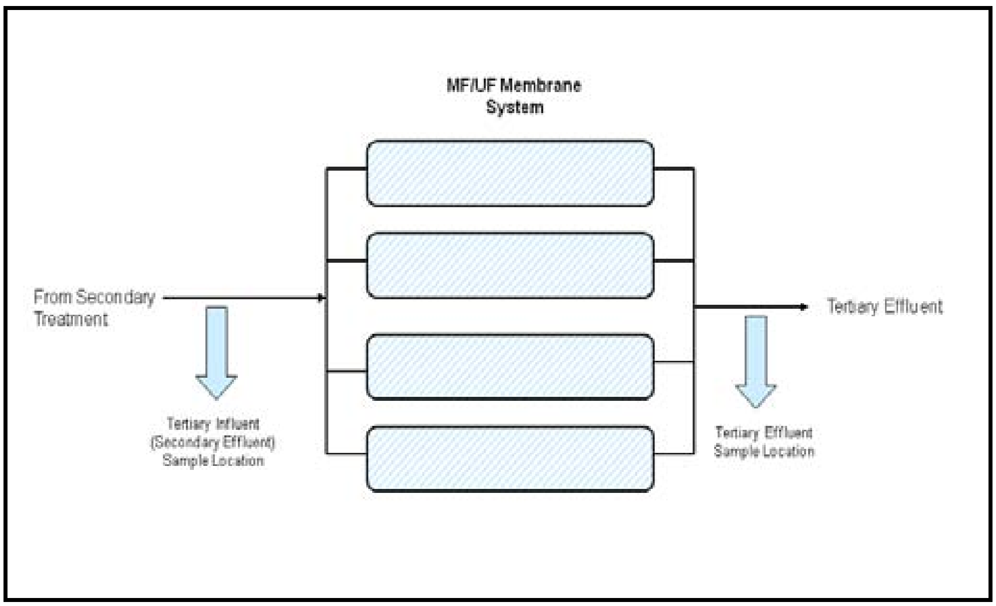

Figure 15-2 - Membrane Filters - Process Schematic and Sampling Locations

In addition to the recommended sample locations and analyses presented in Table 15-4, it is recommended that the following be monitored:

- Feed water and filtrate flow (or volume);
- Operating pressure (transmembrane pressure, TMP);
- Filtrate flux;
- Upstream strainer or screen operation;
- Air flow rate for scouring (if applicable); and
- Volume of chemicals applied to upstream processes or for post-precipitation.

Filtrate flux must be calculated from direct measurements of permeate flow at a given operating pressure. Any changes in flux and recovery rate should be investigated and corrective action should be taken. Feed quality impacts the operating flux and consequently, affects the resulting throughput capacity.

Operation of upstream strainer(s) should be monitored to ensure pressure loss across the strainer does not exceed the design value, as this may reduce feed flow to the membrane process.

Monitoring of chemical use is recommended. Chemical use is a function of the effectiveness of the cleaning system and air scour and backpulse/relaxation operations. Chemical feed rates or volumes should be tracked weekly or monthly to determine if processes upstream of the membrane are working properly or need to be optimized to improve the feed quality to the membrane system.

Table 15-5 presents a range of filtered water quality achieved by MF and UF processes. The data presented are typical values and it should be noted that the composition of the membrane filter feed, especially in terms of soluble constituents, can impact filtrate quality.

Table 15-5 – Membrane Filters – Typical Filtrate Quality for MF and UF Treatment Facilities

(Adapted from WEF, 2006)

Parameter	Range of Values	Removal
BOD (mg/L)	< 2 - 5	85 - 95 %[1]
TOC (mg/L)	5 - 25	5 - 60 %[1]
TKN (mg/L)	5 - 30	6 - 8%[1]
TP (mg/L)	0.1 - 8	1.5 - 3%[1]
Iron (mg/L)	< 0.2	0 - 20%[1]
TSS (mg/L)	Below detection limit	> 99 %
Turbidity (NTU)	< 0.1	95 - 99 %
Fecal Coliform (CFU/100 mL)	< 2 - 100	3 to > 6 log
Virus (PFU/100mL)	< 1 - 300	0.5 to 6 log
Protozoan Cysts (No./100 mL)	< 1	3 to > 6 log
Silt Density Index	< 2 - 3	N/A

Notes:

N/A – Not Available

1. In the absence of chemical treatment, minimal to no removal can be expected if parameter exist only in soluble form.

15.2.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the membrane filtration process are shown in Table 15-6.

Table 15-6 – Membrane Filters – Symptoms and Causes of Common Problems

Problem	Description	Mitigation
Fouling.	• Adsorption or clogging of material on the membrane surface which cannot be removed during the backwash cycle • Cake formation that is sometimes identified as biofilm (Metcalf & Eddy, 2003) • Fouling reduces the recovery rate achieved by the system • Caused by wastewater with metal oxides, organics and inorganics, colloids, bacteria and microorganisms • The rate of membrane fouling can be reduced but it cannot be prevented from occurring over time	• Pretreatment prior to membrane filtration (such as upstream clarification, and/or fine screening) • Membrane backwashing with water and/or membrane scouring with air • Chemical cleaning of membranes • Increase membrane surface scouring or crossflow velocity • Increase amount of membrane surface area to reduce applied flux • Optimization of upstream chemical addition (coagulant and/or polymer)
Scaling.	• Formation of scales or precipitates on the membrane surface • Scaling occurs in wastewater with calcium sulfate, calcium carbonate, calcium fluoride, barium sulfate, metal oxides and silica (Metcalf & Eddy, 2003)	• Preventative cleaning (backwashing or chemical cleaning) • Adjustment of operational variables, recovery rate, pH, temperature • Optimization of upstream chemical (coagulant and/or polymer) addition points
Membrane degradation.	• Gradually with time, membrane degradation is inevitable (Metcalf & Eddy, 2003) • Over time, the flux gradually decreases and less permeate is produced by the membrane (Metcalf & Eddy, 2003)	• To prolong membrane life the following substances should be limited in the feedwater: acids, bases, pH extremes, free chlorine, bacteria and free oxygen (Metcalf & Eddy, 2003) • Eventual replacement of membranes required
Poor effluent quality.	• Increase in turbidity or particle count may indicate damage to membrane, process piping or process seals	• Optimization of upstream processes • An integrity test should be performed to identify possible damaged membranes, process piping or process seals

Table 15-6 – Membrane Filters – Symptoms and Causes of Common Problems (continued)

Problem	Description	Mitigation
Increase or decrease in transmembrane pressure (TMP).	• Either a gradual increase or a sudden drop in membrane pressure is observed • Membrane performance is strongly affected by changes in temperature. At low temperatures, water viscosity increases and membrane permeability decreases	• Gradual TMP increase indicates that a membrane cleaning sequence needs to be initiated • Sudden TMP decrease is a sign of membrane damage • Temperature of the feed water should be monitored

15.2.4 Optimizing Process Performance

Optimization of pretreatment, such as upstream clarification and/or fine screening will increase the efficiency and life of the membranes. Pretreatment requirements will depend on the influent quality and effluent limits. Membrane fouling, backwashing and chemical cleaning frequency can also be minimized through the optimization of pretreatment processes.

Membrane degradation is inevitable. There are measures that can be taken to prolong membrane life. Experience with full scale applications in Ontario have shown that membrane replacement is generally required every 7 to 10 years.

Proper dosing of chemical for removal of phosphorus or other soluble contaminants is vital in order to obtain good quality effluent without overloading the membrane with solids. The addition of chemical coagulants to the feed water will also reduce the membrane fouling potential. Inside-out UF membranes often dose with less than 5 mg/L of ferric chloride to reduce membrane fouling potential. Although overdosing with coagulants will not negatively affect the effluent quality, the amount of chemicals used should be minimized to reduce operating costs. Jar testing is recommended to evaluate the effects of chemical dose on filtered water quality.

Operating membranes at elevated flux levels can increase fouling potential. Routine monitoring of membrane flux is recommended to ensure membrane is operating below the design value at which deterioration of system performance begins to occur. The optimum membrane design flux is normally established during pilot testing and is based on the fouling characteristics of the secondary effluent, the membrane material and the membrane system configuration. Conversely, operating at a reduced flux can result in inefficient use of installed membrane capacity. Membrane modules can be taken off-line or put back on-line to allow operation at an optimum flux for performance and membrane life.

Optimization of backwash frequency will aid in maintaining low transmembrane pressure during operation of the membrane system. However, consideration must be given during optimization to ensure that the increased backwashing does not decrease the overall recovery of water.

Frequency of chemical cleaning is site specific. Reports have shown that membrane facilities treating secondary effluent conduct chemical cleaning once every few days to once every two months (WEF, 2006). The frequency of cleaning will depend on the fouling characteristics of the wastewater, the applied flux, and use of chlorine upstream of the membrane. Generally, it is recommended that membranes are cleaned once every two to four weeks (WEF, 2006).

15.3 BALLASTED FLOCCULATION

15.3.1 Purpose and Types of Ballasted Flocculation Systems

In the ballasted flocculation process, a coagulant or polymer, such as alum, ferric sulfate or anionic polymer, is used with a ballasted material, typically micro-sand (micro-carrier or chemically enhanced sludge can also be used) (MOE, 2008). Wastewater is pumped into a rapid-mix tank and coagulant is added. The ballast material is added to a chemically stabilized and coagulated suspension of particulate solids and, simultaneously, the ballast agent coagulates with the chemical precipitate and particulate solids to form “ballasted” flocs (Young and Edwards, 2000). After flocculation, the suspension is transferred into a sedimentation basin where the ballasted flocs settle. The flocs formed are heavier and larger than conventional chemical flocs and sedimentation can occur ten times faster than with conventional processes (EPA, 2003). A hydrocyclone separates the ballasting agent from the ballasted floc and the ballasting agent is recycled back to the flocculation basin while the sludge is sent for processing and disposal (Young and Edwards, 2000).

The main advantage of the ballasted flocculation process is its ability to handle high loading rates within a small footprint. The ballasted flocculation units are compact, making application of this system possible in a very small space (MOE, 2008). The small footprint of the system will reduce the surface area of the clarifiers and consequently minimize short-circuiting and flow patterns caused by wind and freezing (MOE, 2008). Another advantage of the system is the ability to treat a wider range of flows without compromising removal efficiency.

The main disadvantage of ballasted flocculation systems is the complexity of the process. It typically requires more operational involvement and more complex instrumentation and controls than traditional sedimentation processes or tertiary filtration processes. The use of ballast requires close monitoring of the recycle and the short retention time requires prompt response to provide optimum coagulant dosages with changing conditions (EPA, 2003). Cleaning and replenishment of lost ballasted material (micro-sand or micro-carrier) is required occasionally.

15.3.2 Evaluating Process Performance

To evaluate the performance of the ballasted flocculation system, the influent and effluent water quality parameters that should be monitored are presented in Table 15-7. The amounts of coagulant and polymer used and micro-sand recirculation rates should also be monitored to ensure that the system is functioning properly.

Table 15-7 – Ballasted Flocculation – Minimum Recommended Process Monitoring

Location	Types of Sample/ Measurement	Parameters/ Analyses	Comments
Tertiary Influent (secondary effluent)	Grab and/or Continuous On-line	• Turbidity • TSS • TP • ortho-phosphorus or soluble P • $CBOD_5$ • Temperature • pH	Continuous monitoring is recommended for temperature, pH, and turbidity. Weekly or daily monitoring of all other parameters is recommended.
Tertiary Effluent from Each Clarifier	Grab and/or Continuous On-line	• Turbidity • TSS • TP • ortho-phosphorus or soluble P • $CBOD_5$ • pH	See comments for tertiary influent.

TSS removals of 80 to 95 percent have been achieved with ballasted flocculation operating with overflow rates of 815 to 3,260 L/(m^2·min) (49 m/h to 196 m/h) (MOE, 2008).

Figure 15-3 presents a process schematic of typical ballasted flocculation process, showing recommended sampling locations.

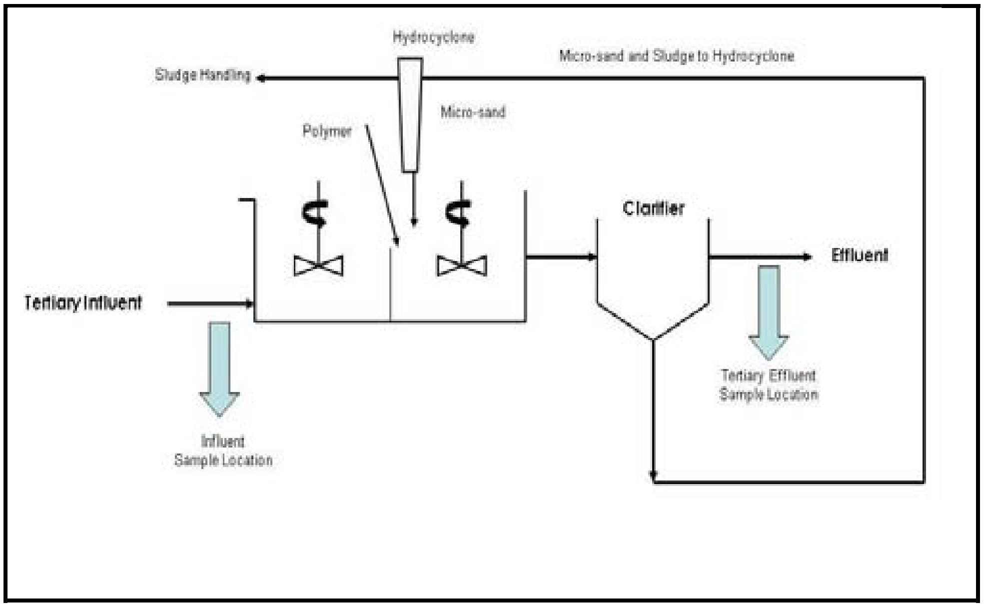

Figure 15-3 - Ballasted Flocculation - Process Schematic and Sampling Locations

15.3.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the ballasted flocculation process are shown in Table 15-8.

Table 15-8 – Ballasted Flocculation – Symptoms and Causes of Common Problems

Problem	Description	Mitigation
Accumulation of organic material on sand particles.	• Occurs when hydroclone malfunctions, causing organic material to accumulate on the sand (USEPA, 2003)	• Does not occur in systems with sludge recycle only (USEPA, 2003)
Floc not forming easily.	• Floc not forming and ballasting agent cannot adhere easily to floc	• There is an optimum amount of ballasting agent associated with each combination of influent turbidity, coagulant dose, and polymer dose (Young and Edwards, 2000)

15.3.4 Optimizing Process Performance

Ballasted flocculation is a proprietary process and there have been few full-scale optimization studies undertaken. Pilot studies are often required to confirm design parameters and optimize operating conditions.

Process variables that can be optimized to enhance the process's effectiveness include:

- Type and concentration of ballasting material (if micro-sand is used varying diameter of sand particles);
- Type, concentration and dose of coagulant and/or polymer;
- Timing of addition of ballasting agent or polymer; and
- Rate of mixing (Young and Edwards, 2000).

It is recommended that the following parameters be monitored during the optimization study:

- Micro-sand/sludge re-circulation pump flow;
- Hydroclone underflow rate;
- Flow rate through sand loss detection boxes; and
- Coagulant and polymer feed rates.

As described in Chapter 16, jar testing should be done to select the optimum chemical(s), dosage(s) and addition point(s).

15.4 CASE HISTORIES

15.4.1 Deseronto WPCP Actiflo™ Pilot Study

The following case study is based on information presented in Gundry (2004).

Background and Objectives

A ballasted sand enhanced sedimentation process (Actiflo™) was commissioned to provide tertiary phosphorus removal at the Town of Deseronto WPCP to meet discharge objectives for the Bay of Quinte. A neighbouring community wanted to extend municipal wastewater servicing which would increase the WPCP rated capacity. The Bay of Quinte Remedial Action Plan (RAP) requires that effluent TP loadings from STPs be maintained at a constant level. Therefore, an increase in sewage treatment plant capacity requires a proportional decrease in the effluent phosphorus concentration limit. In response to RAP requirements, a pilot study was undertaken to optimize the performance and capacity of the enhanced tertiary sedimentation process for phosphorus removal.

The Deseronto WPCP consists of two package-type extended aeration plants with an overall rated capacity of 1,539 m^3/d operating in extended aeration mode and 1,600 m^3/day if operating in the contact stabilization mode. The system was designed based on an Actiflo™ loading rate of 55m/hr at peak flows of 5,478 m^3/d.

Pilot Study

The pilot study used a trailer-mounted Actiflo™ ballasted sand flocculation system with secondary effluent from the secondary clarifier as influent. Various Actiflo™ hydraulic loading rates ranging from 60 to 120 m/hr were tested and corresponding TP and turbidity levels were measured. High removal efficiencies were achieved for TP for all flows tested. Results of the pilot study are shown in Table 15-9.

Table 15-9 – Deseronto WPCP Pilot Study Results

Filter Rate (m/hr)	Total Phosphorus (mg/L)		Turbidity (NTU)	
	Influent	Effluent	Influent	Effluent
60	0.52	0.08	7.0	1.1
80	0.81	0.11	16.7	1.7
100	1.20	0.15	21.2	1.6
120	0.78	0.15	77	2.1

Pilot plant average effluent TP concentrations were 0.15 mg/L, half the compliance limit of 0.3 mg/L, at a loading rate of 120 m/hr which is 220 percent greater than the design loading rate for the full-scale Actiflo™ system.

Alum was added to the pilot plant feed at dosages between 61 and 64 mg/L and Magnifloc 1011 polymer was tested with dosages varying between 0.7 to 1.1 mg/L. It was found that increasing the Magnifloc 1011 polymer dose from 0.7 to 1.0 mg/L improved treated effluent turbidity.

High removal efficiencies were achieved for TP for all flows tested and the pilot study indicated that the design rise can be doubled to 120 m/hr while providing effluent TP concentrations below the objective of 0.3 mg/L. Based on an overall plant effluent TP loading criterion of 0.48 kg TP/d, the plant rated capacity could be increased to 2,400 m^3/d, which is 150 percent of the design rated capacity of the STP.

15.4.2 Dundas WPCP Sand Filter Optimization

The following case study is based on information presented in Enviromega Limited (1992).

The Dundas WPCP, which discharges into the Hamilton Harbour Area of Concern, is required to achieve an annual effluent total suspended solids concentration below 5 mg/L.

The performance of the two sand filters at the Dundas WCPC at various loading conditions was monitored using off-line sampling and on-line instrumentation. The low-rate filters are operated in parallel with a design filtration rate of 74 m/d at a design flow of 18,200 m^3/d. The filters consisted of a single media (sand) in the size range of 600 to 850 µm. Alum is injected upstream of the secondary settlers for phosphorus removal. Backwashing of the filters is initiated when the pressure drop through the filter reaches a pre-set level.

The filters were evaluated under normal and elevated hydraulic and solids loading conditions. To achieve elevated hydraulic loading, one of the two filters was taken off-line. Elevated solids loading was achieved by pumping mixed liquor directly into the filter influent channel.

Plant influent flowrate, filter liquid head level, backwash cycle frequency, turbidity, BOD_5, TP and TSS were monitored. Results of the study are presented in Table 15-10.

Table 15-10 - Dundas WPCP Tertiary Filtration Study Results: TSS, BOD_5, TP

Phase	Filter Rate (m/d)	TSS (mg/L)		BOD_5 (mg/L)		Total Phosphorus (mg/L)	
		Influent	Effluent	Influent	Effluent	Influent	Effluent
1. Baseline	61	8.3	1.2	5	3	0.5	0.1
2. Hydraulic Load	109	9.6	1.7	4	2	0.3	0.1
3. Solids Load	48	27.8	3.2	8	3	1.0	0.1
4. Hydraulic and Solids Loads	92	18.2	3.0	7	3	1.0	0.1

Filter effluent quality was good under all loading conditions evaluated.

The mean daily average filter effluent TSS concentration was 1.2 mg/L under the Phase 1 baseline conditions. The BOD_5 and TP effluent concentrations were 3 mg/L and 0.1 mg/L, respectively.

The Phase 2 mean daily average effluent TSS results at the elevated hydraulic loading rate increased slightly to 1.7 mg/L. The BOD_5 and TP effluent concentrations were 2 mg/L and 0.1 mg/L, respectively.

During Phase 3 (elevated solids loading rate), the mean daily filter effluent TSS concentration was 3.2 mg/L, indicating that as the influent TSS concentration increased, the effluent TSS concentration also increased. The BOD_5 and TP effluent concentration remained relatively constant at 3 mg/L and 0.1 mg/L, respectively.

During Phase 4 (elevated hydraulic and solids loading), the mean daily filter effluent TSS concentration was 3 mg/L, again indicating that as the influent TSS concentration increased, the effluent TSS concentration also increased. The BOD_5 and TP effluent concentration remained about the same at 3 mg/L and 0.1 mg/L, respectively.

During each phase of testing, the average number of hours the backwash system was operated was monitored. The duration and frequency of backwash cycles for each loading condition is presented in Table 15-11. Generally, the total number of backwash hours increased with an increase in solids loading. During Phase 4, filter bypass was observed during peak loading rates indicating that the plant could not accommodate the plant design flow rate of 18,200 m^3/d under these loading conditions.

Table 15-11 – Dundas WPCP Tertiary Filtration Study Results - Backwash

Phase	Duration[1] (h/d)	Frequency[2]
1. Baseline	6.3	3
2. Hydraulic Load	8.4	5
3. Solids Load	9.5	4
4. Hydraulic and Solids Loads	10.2	> 7[3]

Notes:

1. Backwash duration in hours per day, including both beds.
2. Number of cycles during the peak period of 8:00 to 16:00 h (both beds).
3. The filter was continuously backwashed from 9:30 to 14:00 h and filter bypass occurred.

15.5 REFERENCES

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CHAPTER 16

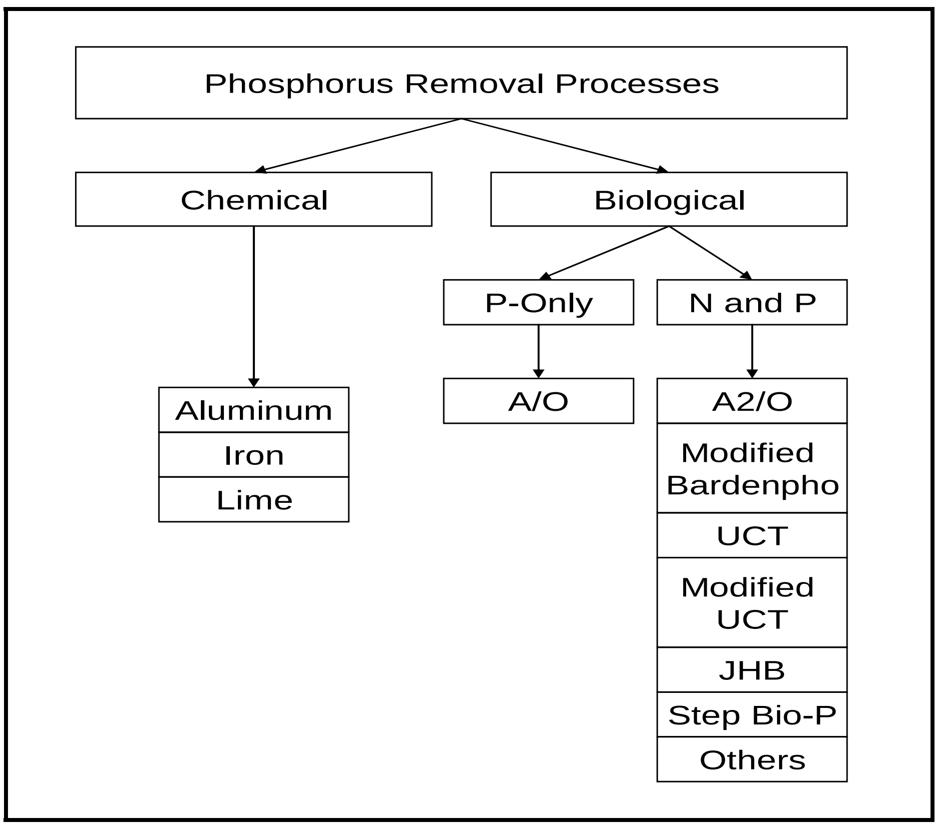
PHOSPHORUS REMOVAL

CHAPTER 16

PHOSPHORUS REMOVAL

16.1 APPROACHES TO ACHIEVE PHOSPHORUS REMOVAL

Removal of phosphorus is achieved by converting dissolved phosphates into solids and then physically separating the solids from the liquid stream. The phosphates are converted into either chemical precipitates or biological solids (microorganisms). The following sections will review the chemical and biological approaches to phosphorus removal as well as the physical methods for the subsequent solids separation process. Numerous processes have been developed to reduce the effluent concentration of phosphorus. In general, these can be divided into two categories, namely chemical phosphorus removal (Section 16.1.1), and biological phosphorus removal (Section 16.1.2 and Section 12.5). Figure 16-1 presents an overview of the common phosphorus removal processes which are discussed within this chapter.



Note: Further information on biological processes available in Section 16.4.1.

Figure 16-1 - Summary of Phosphorus Removal Processes

16.1.1 Chemical Phosphorus Removal

Chemical phosphorus removal involves the addition of chemicals that react with ortho-phosphorus to form insoluble precipitates. Chemicals commonly used for chemical phosphorus removal are aluminum sulphate (alum), iron salts and lime. These metals salts contain the multivalent ions of aluminum [Al(III)], iron [Fe(II) or Fe(III)], and calcium [Ca(II)] which react with soluble phosphorus to form the insoluble precipitate (Metcalf & Eddy, 2003). To assist the formation and increase in the size of the solid particles, polymers can also be added to improve the solids separation process. More details on the mechanisms involved in chemical phosphorus removal are provided in Section 16.3.

16.1.2 Biological Phosphorus Removal

Biological phosphorus removal occurs as a result of microorganisms which uptake phosphorus during biomass synthesis in excess of their metabolic requirements. The ability of some microorganisms to uptake phosphorus was first discovered in the 1960s-1970s (Nutt, 1991). It was shown that sequentially exposing the mixed liquor to anaerobic conditions followed by aerobic conditions resulted in the selection and conditioning of microorganisms that accumulate higher levels of phosphorus within the cells than in conventionally operated activated sludge processes (WEF *et al.*, 2005).

These microorganisms known as phosphate accumulating organisms (PAOs) are able to take up excess phosphorus as well as assimilate and store volatile fatty acids (VFAs) under anaerobic conditions (MOE, 2008). Energy produced through hydrolysis of polyphosphates previously stored within the bacterial cell results in the release of phosphorus from the microorganisms to the surrounding environment (WEF *et al.*, 2005). Sequentially, when the microorganisms are introduced to an aerobic environment, the stored organic substrate is consumed and the organisms take up the orthophosphates that were hydrolyzed and released previously (WEF *et al.*, 2005). Within the cell, the phosphorus is stored in solid granules which can be removed from the systems by separating out the cells (WEF *et al.*, 2005).

Section 16.4 discusses the process configurations that result in biological phosphorus removal within secondary biological suspended growth sewage treatment processes. Biological phosphorus removal is also discussed in Chapter 12.

16.2 SOLIDS SEPARATION REQUIRED FOR CHEMICAL AND BIOLOGICAL PHOSPHORUS REMOVAL

Phosphorus removal, whether by chemical or biological methods, is a two-step process - the initial conversion of the soluble phosphorus into particulate phosphorus and the subsequent removal of the particulate phosphorus. To optimize the process, the efficiency of each step must be optimized. The type of phosphorus removal utilized will impact the nature of the solids generated as chemical phosphorus removal results in a chemical floc which can have different characteristics than the biological floc produced during biological phosphorus removal. There are a number of processes that can be employed to effectively remove solids including those presented in Table 16-1.

Table 16-1 – Solids Separation Mechanisms Utilized for Phosphorus Removal

(WERF, 2008b)

Solids Separation Process(es)	Specific Processes
Conventional Sedimentation	• Primary clarifier • Secondary clarifier
Filtration	• Single media filter • Dual media filter • Multi-media filter • Deep bed filter • Cloth media filter
High Rate Sedimentation	• Ballasted sedimentation • Solid blanket (contact) clarifiers • Tube settlers
Two-stage Filtration	• Two filter units in series
Microfiltration or Ultrafiltration	• Membrane filtration
Magnetic Based Separation	• Ballasted separation and magnetic polishing step

The type of solids separation process utilized depends on the level of phosphorus removal required, the sewage characteristics and the type of treatment process. Details on optimization of the solids separation process, whether by conventional clarification processes or in tertiary processes, can be found in Chapters 11, 14 and 15.

16.3 OVERVIEW OF CHEMICAL PHOSPHORUS REMOVAL

Chemical phosphorus removal is achieved by adding metal cations that are able to transform the orthophosphate in the sewage into an insoluble precipitate. There are a number of factors influencing the efficiency of chemical phosphorus removal including the type of chemical employed, the dosage of that chemical, the location(s) at which the chemical(s) is added to the sewage treatment stream, the sewage characteristics, whether a polymer is added to assist in solids flocculation, and the type of solids separation processes utilized.

16.3.1 Chemicals

Below is a brief summary of the most common chemicals associated with chemical phosphorus removal. Additional information on chemicals associated with phosphorus removal can be found in MOE (2008), WEF/ASCE/EWRI (2005) and EPA (2000).

There are three forms of aluminum compounds that are typically used for phosphorus precipitation: aluminum sulphate [$Al_2(SO_4)_3 \cdot 14H_2O$], sodium aluminate ($Na_2Al_2O_4$), and polyaluminum chloride ($Al_nCl_{3n-m}(OH)_m$). Sodium aluminate is used for process water requiring additional alkalinity and polyaluminum chloride is used for enhanced solids removal. Of these forms, aluminum sulphate (alum) is the most commonly used.

There are four iron salts that may be used for phosphorus precipitation: ferrous chloride ($FeCl_2$), ferrous sulphate ($FeSO_4$), ferric chloride ($FeCl_3$) and ferric sulphate [$Fe_2(SO_4)_3$]. Of these iron salts, ferric chloride is the most commonly used in Ontario.

Lime is currently not widely used in North America for a number of reasons including: chemical handling safety concerns, high chemical dosage requirements and high sludge generation rates. In addition, high calcium concentrations from lime addition can inhibit volatile suspended solids (VSS) destruction during digestion and limit disposal options.

In addition to the above-mentioned chemicals, hybrid chemicals that contain a combination of alum and iron could also be considered. These chemicals include acidified alum and alum/iron salt mixtures. Further information on hybrid chemicals can be obtained through chemical suppliers.

The addition of these chemicals can impact the alkalinity levels available for biological treatment. In nitrifying plants, low alkalinity can inhibit nitrification (Chapter 12). Consideration should be given to the impacts of chemical addition on available alkalinity for STPs utilizing chemical phosphorus removal.

Table 16-2 presents the theoretical stoichiometric dosing ratios required for phosphorus removal using aluminum and iron salts. As the dosing required varies depending on a number of factors including the sewage characteristics, chemical usage, and addition point(s), the actual chemical doses should be determined using jar tests (Section 16.3.6) and confirmed by full-scale trials. Figure 16-2 illustrates the impact of point addition on the effectiveness of phosphorus removal using aluminum. Other factors that should be taken into account when choosing a chemical to use are the sludge generation rate associated with the chemical, cost, availability of chemical, safety and ease of handling.

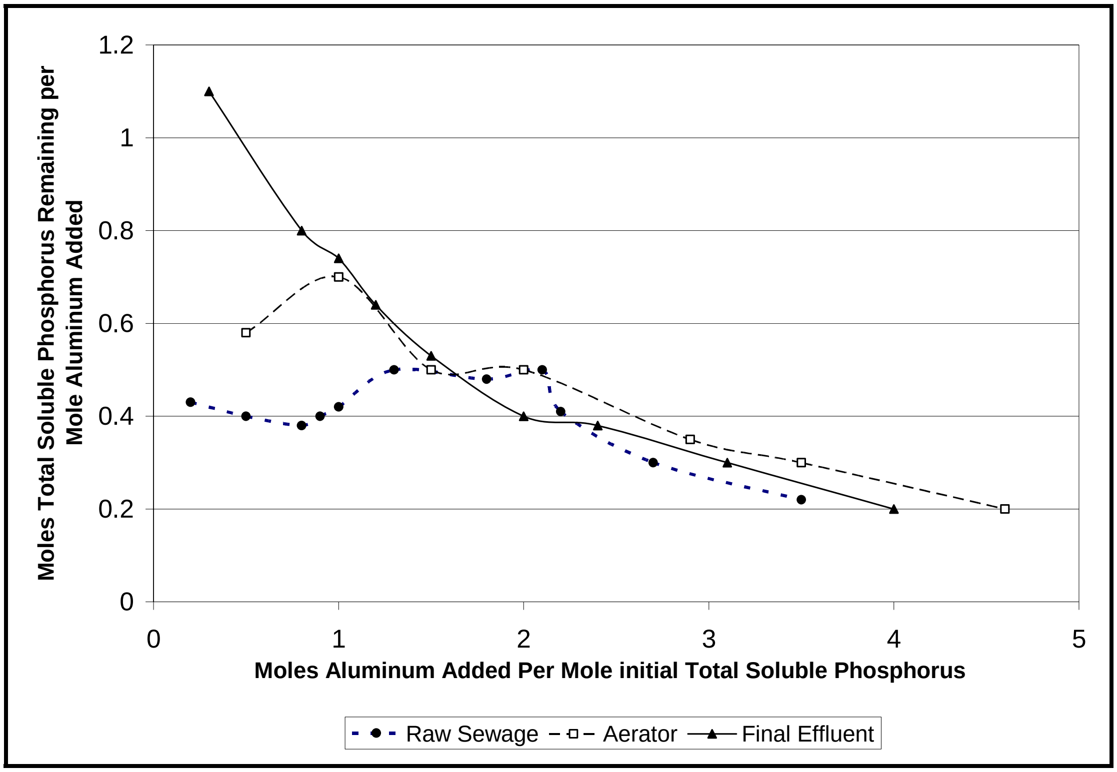


Figure 16-2 – Impact of Point Addition on Effectiveness of Phosphorus Removal Using Aluminum

Table 16-2 – Stoichiometric Weight Ratios for Metal Salts

(Adapted from WEF *et al.*, 2005)

Chemical Name		Molecular Weight	Weight Ratio (Chemical:P)
Aluminum Sulphate	$Al_2(SO_4)_3 \cdot 14H_2O$	594	10:1
Sodium Aluminate	$Na_2O \cdot Al_2O_3 \cdot 3H_2O$	218	4:1
Polyaluminum Chloride	$Al_nCl_{3n-m}(OH)_m$	134	4:1
Ferric Chloride	$FeCl_3$	162	5:1
Ferric Sulphate	$Fe_2(SO_4)_3$	400	7:1
Ferrous Chloride	$FeCl_2$	127	6:1
Ferrous Sulphate	$Fe(SO_4)$	152	7:1

16.3.2 Flocculants

Polymers are a flocculation aid, typically used in conjunction with metal salts to facilitate the removal of particulate phosphorus produced by the precipitation reactions. They may be used where solids do not flocculate or settle well in secondary clarifiers. Polymers may also be added with metal salts to the primary clarifiers to further enhance TSS, particulate BOD_5 and TP removal, thereby reducing solids and organic loading to the downstream bioreactors (Section 11.2).

Polymers are proprietary and information relating to dosage and composition are specific to each product. Each manufacturer of polymers should provide information regarding the expected dosage required, but jar testing should be performed in order to optimize the polymer requirements for each application (Section 16.3.6).

16.3.3 Chemical Addition Points

Chemical addition for phosphorus removal typically occurs at three different locations in a sewage treatment plant. Phosphorus removal in the primary clarifiers is known as chemically enhanced primary treatment (CEPT) which is a type of pre-precipitation (Section 11.2). Co-precipitation (or simultaneous precipitation) is the removal of phosphorus in the secondary clarifiers via the addition of chemical precipitants to the bioreactor mixed liquor, or to the bioreactor effluent (secondary clarifier influent). Post-precipitation is the addition of coagulants to secondary clarifier effluent for phosphorus removal in tertiary treatment processes. A fourth mode of operation involving the addition of chemicals to multiple points in the sewage treatment train exists, called multi-point addition.

Figures 16-3 to 16-6 present process schematics for various chemical phosphorus removal processes, along with recommended sampling locations for each process. For simplicity, the solids separation process included in all figures is secondary clarification; however, as mentioned in Section 16.2, there are numerous processes that can be used for tertiary solids separation.

Pre-precipitation

Pre-precipitation can be effective in removing phosphorus. It also has the added benefit of increasing solids and BOD_5 removal in the primary clarifiers, reducing the organic loading to the downstream bioreactors (MOE, 2008). Figure 16-3 presents a process schematic for pre-precipitation chemical addition for phosphorus removal. When implementing pre-precipitation, it is important to ensure that adequate phosphorus is available in the primary effluent to support the biological activity in the downstream process stage. More detailed information on CEPT is available in Section 11.2.

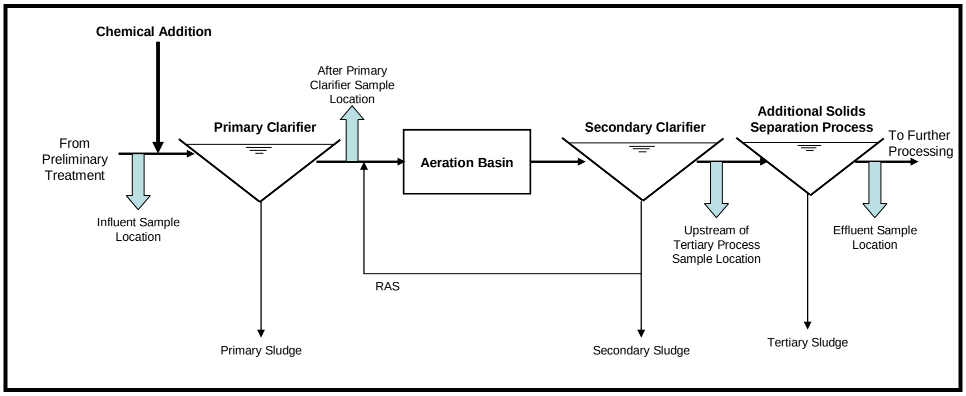


Figure 16-3 – Process Schematic and Recommended Sampling Locations for Pre-Precipitation

Co-Precipitation or Simultaneous Precipitation

Co-precipitation, or simultaneous precipitation, is the most common mode of phosphorus removal in Ontario (MOE, 2008). Chemicals are typically added at the end of the aeration basin to ensure adequate contact between the chemical and the mixed liquor. Since phosphorus removal often occurs downstream of the bioreactors, nutrient deficiency is not a concern, and chemical dosing does not have to be as closely monitored to ensure a healthy biomass is maintained in the bioreactors. As well, when dosing alum or ferric salts, it has been found that chemical addition for phosphorus removal to the aeration tank effluent can require 35 percent less chemical than dosing to the aeration tank influent (MOE, 2008).

Figure 16-4 presents a process schematic for co-precipitation coagulant addition for phosphorus removal.

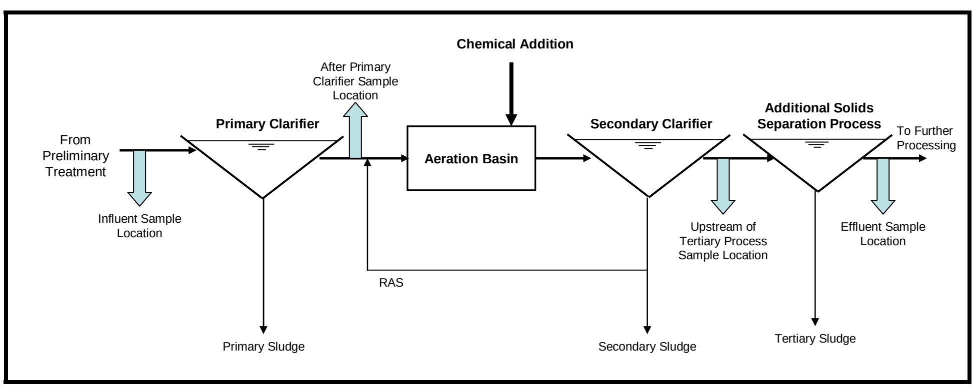


Figure 16-4 - Process Schematic and Recommended Sampling Locations for Co-Precipitation

Post-Precipitation

Post-precipitation involves the addition of coagulants upstream of a tertiary solids separation unit. There are few plants in North America that use only post-precipitation. It is normally used as part of a multi-point addition operating scheme. Alum or iron salts are more commonly used over lime for post-precipitation due to operational difficulties associated with lime handling, high sludge production, as well as the need for pH adjustment and recarbonation. Phosphorus removal after the bioreactor can allow for low TP while removing any risk to the biological processes (Nutt, 1991; MOE, 2008).

Figure 16-5 presents a process schematic for post-precipitation coagulant addition for phosphorus removal.

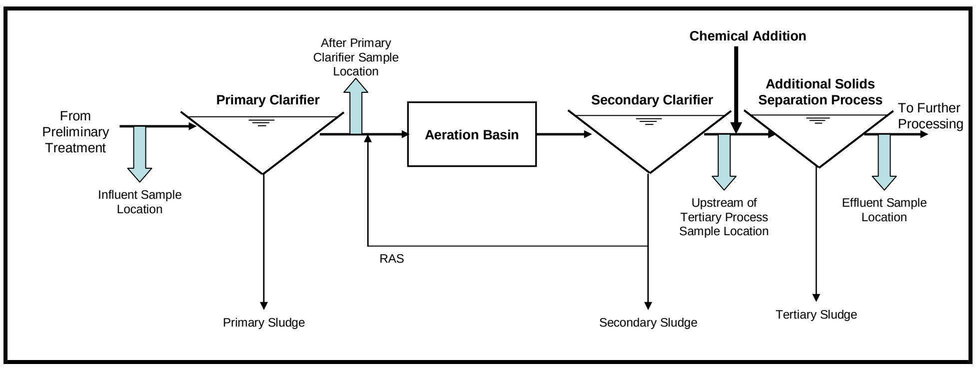


Figure 16-5 – Process Schematic and Recommended Sampling Locations for Post-Precipitation

Multi-Point Addition

Multi-point chemical addition provides additional flexibility and control of the phosphorus removal process and can produce an optimal operating point with respect to chemical dosages and sludge production. Typically, multi-point addition implies addition of chemicals at two points (dual-point) or at three points (triple-point). Equivalent phosphorus removal may be achieved with as much as 20 percent less coagulant using multi-point addition (WEF *et al.*, 2005). In addition, the effluent TSS from plants practising multi-point addition will contain lower levels of phosphorus as some of the phosphorus has been removed upstream of the bioreactor. Hence, the effluent will have a lower TP concentration at the same soluble P and TSS concentrations.

Figure 16-6 presents a process schematic for multi-point chemical addition for phosphorus removal.

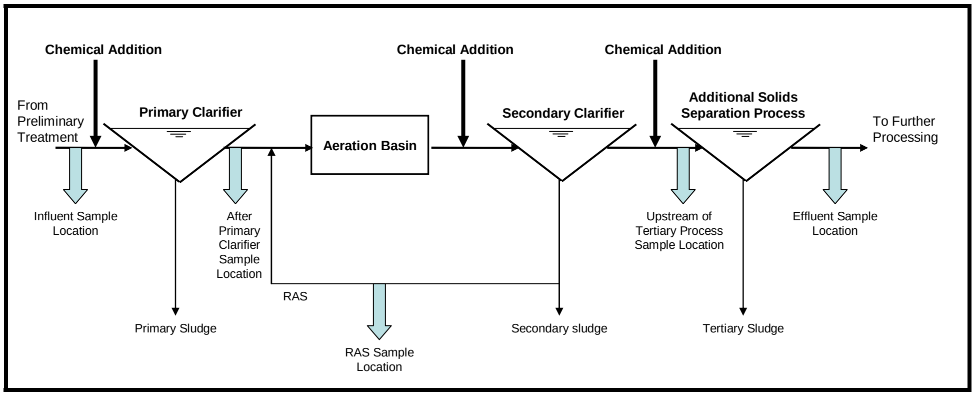


Figure 16-6 – Process Schematic and Recommended Sampling Locations for Multi-Point Addition of Chemicals

16.3.4 Evaluating Process Performance

Table 16-3 presents monitoring recommendations, in terms of sampling locations and analyses, in order to evaluate the performance of the chemical phosphorus removal process.

Table 16-3 – Recommended Process Monitoring to Evaluate Chemical Phosphorus Removal Performance

Sample Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Influent	Composite Recommended	• Flow rate • TSS • TP • Orthophosphate • pH • Alkalinity	
After Primary Clarification	Composite Recommended	• TSS • TP • Orthophosphate • pH • Alkalinity	Primary effluent samples should be collected on the same days as primary influent samples so that removal efficiencies across the primary clarifiers can be calculated. Online equipment can be used to continuously measure orthophosphate concentration for process control.

Table 16-3 - Recommended Process Monitoring to Evaluate Chemical Phosphorus Removal Performance (continued)

Sample Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Upstream of Tertiary Process	Composite Recommended	• TSS • TP • Orthophosphate • pH • Alkalinity	Although the secondary effluent concentrations of these parameters are affected by the upstream biological treatment process, these values can also provide insight into secondary clarifier performance. If the performance of a particular clarifier is suspect, effluent samples from individual clarifiers can be collected for comparison purposes.
Final Effluent	Composite Recommended	• TSS • TP • Orthophosphate • pH • Alkalinity	Online equipment can be used to continuously measure orthophosphate concentration for process control.

In addition to the recommended sample locations and analyses presented in Table 16-3, it is recommended that the following also be monitored:

- Quantities and characteristics of coagulants and polymers added at each addition point;
- Chemical-to-phosphorus ratio (Al:P, Fe:P);
- Performance of system in terms of nitrogen removal;
- Mixing intensity, velocity gradients (G) and contact time (τ) at the addition point(s) which is especially important for the post-precipitation addition point; and
- Volume of sludge produced from the system.

16.3.5 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the chemical phosphorus removal process that result in higher than expected effluent TP concentration are presented in Table 16-4.

Table 16-4 – Chemical Phosphorus Removal – Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Poor removal of precipitated phosphorus solids.	• High TP and low orthophosphate concentrations in the effluent • High TSS in the effluent	• Solids removal efficiency in solids separation stage lower than typical values (Chapter 11, 14 and/or 15) • Undersized solids separation process (i.e. for surface area, HRT or height) • Floc destruction due to high turbulence prior to solids separation unit • Flow patterns within solids separation unit allow for re-suspension of solids
Poor orthophosphate removal.	• Orthophosphate concentration remains relatively high from influent to effluent	• Mechanical failure of chemical dosing or mixing equipment • Chemical dosing not correct (Section 16.3.6) • Inadequate contact time between chemical and sewage (Section 16.3.6) • Incomplete or inefficient mixing of chemical and sewage (Section 16.3.6) • pH not in optimum range for chemical used • Sudden change in influent characteristics
High sludge production.	• Higher than expected sludge production	• Jar testing should be performed to ensure optimal chemical selection, dosage, mixing and contact time (Section 16.3.6)

16.3.6 Jar Testing

Jar tests should be utilized to determine the optimal chemical dosage for phosphorus removal. Due to the variability of sewage composition, the determination of the amount of chemicals required to achieve phosphorus removal through chemical precipitation based on theoretical stoichiometry ignores competing reactions. In addition, the contact time and mixing required to achieve the desired effluent phosphorus concentration can be determined through jar testing. Factors which can affect chemical dosage requirements include pH, alkalinity, trace elements, and ligands in the sewage (Metcalf & Eddy, 2003; EPA, 1987). Also, the addition of polymer can impact the dosage of chemicals required for phosphorus removal. For these reasons, laboratory jar testing is recommended to ensure appropriate chemical dosage, mixing and contact time. Laboratory bench-scale jar testing equipment is widely available from scientific equipment suppliers and typically allows six one-litre samples to be tested concurrently. A central controller mixes all samples at the same rate and length of time which allows all but one factor (i.e. polymer or coagulant dosage) to remain constant (EPA, 2000). Figure 16-7 presents an image of a jar testing apparatus.

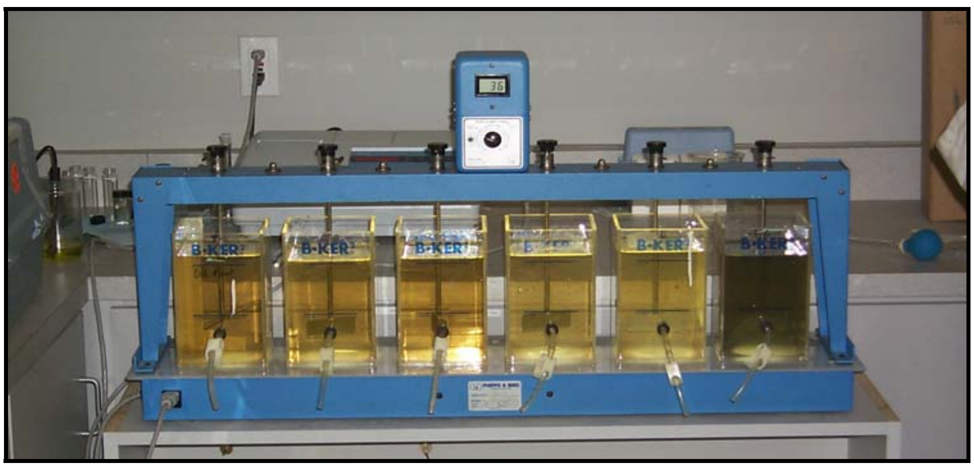


Figure 16-7 – Jar Testing Apparatus

Jar testing result can lead to inconclusive or incorrect results as a result of the following (EPA, 2000; WERF 2008a):

- small dosages of the chemicals (i.e. 1-2 ml of stock solution could result in 1-20 percent error when scaled to full-scale chemical dosages);
- dosage of chemicals inconsistent;
- utilizing old chemicals;
- mixing, flocculation and solids separation that differ from that expected in the full-scale facility;

- narrow dosage range of chemicals; and
- different people performing the tests.

Jar testing results typically over-estimate the precipitant dosages actually required at full-scale. This is particularly true for simultaneous precipitation (co-precipitation) because the metal salt that accumulates in the mixed liquor remains reactive, resulting in improved phosphorus removal at lower dosages than predicted by jar tests. For more information on jar testing, reference should be made to Weighand and Weighand (2002) and Clark and Stephenson (1999).

16.3.7 Optimization of Chemical Phosphorus Removal

Phosphorus removal is a two-step process in which soluble phosphorus is first converted to a particulate form and the particulate phosphorus is then removed by a solids separation process. Optimization of the solids separation processes are discussed in Chapters 11, 14 and 15. The chemical phosphorus removal process typically involves the use of laboratory jar testing as discussed in Section 16.3.6.

Table 16-5 provides the possible variables that can impact chemical precipitation and ways to optimize the removal process.

Table 16-5 – Optimization of Chemical Phosphorus Removal

Parameter	Methods to Optimize
Chemical(s) Used	Use jar testing (Section 16.3.6) to test a variety of metal salts and/or polymers in order to ensure that the most appropriate combination of chemicals are utilized for the process and sewage characteristics.
Chemical Addition Location(s)	In processes in which a single-point addition is utilized, adding chemicals at multiple locations could improve phosphorus removal as well as lower chemical usage. Full-scale testing is the best method to determine the appropriate locations for chemical additions. The optimum locations are dependent on the treatment plant configuration.
Chemical Dosage	Jar testing (Section 16.3.6) should be used to predict the most appropriate chemical doses. For multi-point chemical addition, the jar testing should reflect the sewage characteristics and conditions at the chemical addition location.
Control of Chemical Dosing	For sewage treatment plants with high variability in flows, chemical dosing by flow-pacing can be utilized to ensure that the applied chemical dosages are not over or under the required amount. Flow proportioning of chemical feed rates can reduce chemical costs and sludge production significantly. In addition, online equipment can be used to continuously measure orthophosphate concentration for process control.
Chemical Contact Time	Use jar testing (Section 16.3.6) to optimize the contact time required between the waste stream and the chemical to ensure optimal precipitation, flocculation and settling of the precipitated solids.

Table 16-5 – Optimization of Chemical Phosphorus Removal (continued)

Parameter	Methods to Optimize
Mixing	Use jar testing (Section 16.3.6) to test the current mixing regime to ensure that it is optimized. A high mixing intensity can result in floc destruction and a low mixing intensity can result in poor contact between the waste stream and chemicals.
pH	Adjustment of the pH could be required to ensure that the pH does not have an impact on phosphorus precipitation. The impact of pH on precipitation depends on the addition point as well as the chemicals used. As an example, alum use in the secondary effluent is optimal at a pH of about 6 (Nutt, 1991).

16.4 OVERVIEW OF BIOLOGICAL PHOSPHORUS REMOVAL

Biological phosphorus removal is achieved by integrating phosphorus into the cell mass in excess of metabolic requirements. Biological phosphorus removal process requires the presence of phosphate accumulating organisms (PAOs) that can take up amounts of orthophosphates in excess of biological requirement when exposed to specific environmental conditions. By exposing PAOs first to anaerobic conditions with adequate levels of readily available organic carbon, they are able to release stored phosphorus. If the PAOs are then exposed to aerobic conditions, they are able to take up phosphorus in excess of their biological growth requirements. Once the orthophosphates have been accumulated within the PAOs, solids separation and sludge wasting are used to ensure that the phosphates are removed from the effluent stream. Further information on the mechanisms involved in biological phosphorus removal can be found in MOE (2008) and WEF *et al.* (2005).

16.4.1 Alternative Process Configurations

There are a variety of processes that have been developed to achieve biological phosphorus removal. All biological phosphorus removal processes include key common process requirements: the mixed liquor must be sequentially exposed to an anaerobic zone followed by an aerobic zone (Nutt, 1991). In some biological phosphorus processes, the removal of nitrogen is also incorporated, which requires the inclusion of an anoxic zone between the anaerobic and aerobic zones. Processes which twin phosphorus and nitrogen removal are called biological nutrient removal (BNR) processes.

One commonly utilized biological phosphorus removal process is the modified Bardenpho process. This process is able to achieve simultaneous nitrogen and phosphorus removal and therefore is also considered to be a BNR process (WEF *et al.*, 2005). The process flow includes five separate zones in sequence (anaerobic, anoxic, aerobic, anoxic, aerobic). As in the majority of BNR processes, there is an internal recycle line for nitrate at the beginning of the first anoxic zone to ensure that nitrates do not enter the anaerobic zone which would impair phosphorus release (WEF *et al.*, 2005).

There are also a number of other biological processes developed to remove phosphorus including: Phoredox (A/O), Anerobic/Anoxic/Oxic (A^2/O), Modified Bardenpho, Step Bio-P, University of Cape Town (UCT), and the Johannesburg processes. For further information on biological phosphorus removal processes, reference should be made to MOE (2008), WEF *et al.* (2005) and Crawford *et al.* (2000).

16.4.2 Evaluating Process Performance

Table 16-6 presents monitoring recommended, in terms of sampling locations and analyses, in order to evaluate the performance of biological phosphorus removal processes.

Table 16-6 – Recommended Process Monitoring to Evaluate Performance of Biological Phosphorus Removal

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Secondary Influent	Composite Recommended	• Flowrate • TSS • BOD_5 • COD • VFAs • TP • Orthophosphate • pH • TKN • TAN • NO_3^-–N • NO_2^-–N	
Within Each Zone (i.e. anaerobic, anoxic, aerobic)	Well Mixed Grab Sample Under Typical Operating Conditions	• DO • Temperature • ORP • pH • TSS • BOD_5	Determination of the location to sample will depend on the type of mixing and location of recycle line input(s).

Table 16-6 – Recommended Process Monitoring to Evaluate Performance of Biological Phosphorus Removal (continued)

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
		• TP • Orthophosphate • TKN (recommended) • TAN (recommended) • NO_3^-–N • NO_2^-–N	
Upstream of Solids Separation	Composite Recommended	• TSS • BOD_5 • TP • Orthophosphate • TKN (recommended) • TAN (recommended) • NO_3^-–N • NO_2^-–N	If the performance of a particular clarifier is suspect, effluent samples from individual clarifiers can be collected for comparison purposes.
Nitrate Recycle Line (if applicable)	Composite Recommended	• Flowrate • TSS • TP • Orthophosphate • TKN (recommended) • TAN (recommended) • NO_3^-–N • NO_2^-–N	

Table 16-6 – Recommended Process Monitoring to Evaluate Performance of Biological Phosphorus Removal (continued)

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Mixed Liquor Recycle Line (if applicable)	Composite Recommended	• Flowrate • TSS • TP • Orthophosphate • TKN (recommended) • TAN (recommended) • NO_3^-–N • NO_2^-–N	
Effluent	Composite Recommended	• $CBOD_5$ • TSS • TP • Orthophosphate • TAN • NO_3^-–N • NO_2^-–N • pH	

In addition to the recommended sample locations and analyses presented in Table 16-6, it is recommended that the following also be monitored:

- Quantity and quality of WAS discharged;
- HRT and SRT in each biological zone; and
- Quantity of chemicals (i.e. for coagulation or scum control) added upstream of the biological phosphorus removal process, if applicable.

Figure 16-8 presents a schematic of the UCT process along with recommended sampling locations. This schematic can also be utilized to determine the sampling locations for the majority of biological phosphorus removal processes. For simplicity, the solids separation process shown is sedimentation; however, as mentioned in Section 16.2 there are numerous processes for solids separation.

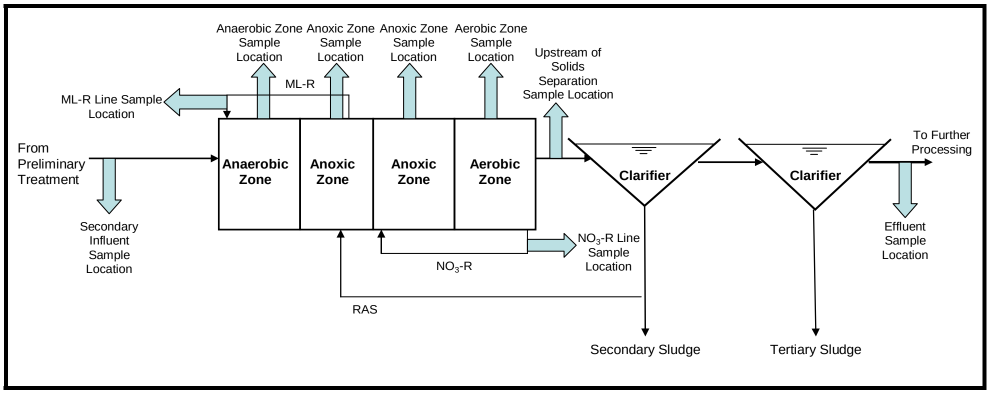


Figure 16-8 – UCT Process Schematic and Recommended Sampling Locations

16.4.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the biological phosphorus removal processes that can result in higher than expected effluent TP concentrations are summarized in Table 16-7. More detailed information on common problems and potential process impacts can be found in Benisch *et al.* (2004).

Table 16-7 - Biological Phosphorus Removal – Common Problems and Potential Causes

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Poor removal of precipitated phosphorus solids.	• High TP and low orthophosphate concentrations in the effluent • High TSS in the effluent	• Solids removal efficiency in solids separation unit lower than typical values (Chapters 11, 14 and/or 15) • Undersized solids separation process (i.e. for surface area, HRT or height) • Floc destruction due to high turbulence prior to solids separation unit • Flow patterns within solids separation unit allow for re-suspension of solids

Table 16-7 - Biological Phosphorus Removal - Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low substrate availability for biological phosphorus removal.	• Low BOD:P ratio (below 20:1) • Orthophosphate below ~1-2 mg/L prior to aerobic zone	• Low concentration of readily available organics in the influent (Section 16.4)
Sudden drop in phosphorus removal and nitrogen removal that occurs suddenly and recovery occurs gradually.	• Rapid decrease in the TP/orthophosphate and nitrogen removal rate in the biological treatment process • Recovery of process over a period of time • Sudden increase in flowrate • Recovery of TP and nitrogen removal rate over a period of time • Drop in TSS within the zones of the biological treatment processes which coincides with increase in effluent TSS	• Chemical inhibition caused by abrupt change in influent sewage characteristics • MLSS loss due to wet weather flow (Chapter 7)
Low phosphorus removal and nitrogen removal within the biological treatment process.	• Low removal rates of TP and TN within the secondary treatment process (i.e. little change in TP and TN between secondary influent and upstream of solids separation)	• Low HRT within biological process due to short-circuiting (Chapter 12) • Long HRT within anaerobic zone
Lower than expected nitrogen removal and phosphorus removal in the biological treatment process.	• No release of orthophosphate within the anaerobic zone (i.e. no increase in orthophosphate between secondary influent and anaerobic zone)	• Low readily available organics for release of phosphorus (Section 16.4) • Low HRT within anaerobic zone
	• No release of orthophosphate within the anaerobic zone (i.e. no increase in orthophosphate between secondary influent and anaerobic zone) • High levels of NO_3^--N within anaerobic zone	• Competition for readily biodegradable material (VFAs) between microorganisms involved in phosphorus removal and denitrification (Chapter 12 and Section 16.4) • High concentration within the recycle stream

Table 16-7 - Biological Phosphorus Removal - Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
	• No release of orthophosphate within the anaerobic zone (i.e. no increase in orthophosphate between secondary influent and anaerobic zone) • Low temperature within biological process zones	• Biological process inhibited by low liquid temperatures (Chapter 12)
Lower than expected phosphorus removal in the biological treatment process.	• No release of orthophosphate within the anaerobic zone (i.e. no increase in orthophosphate between secondary influent and anaerobic zone) • DO present within the anaerobic zone	• DO carryover from aerated grit removal (Chapter 10) or due to wet weather flow (Chapter 7)
	• High orthophosphate leaving anaerobic zone (i.e. large increase in orthophosphate) • Increase in BOD concentration within anaerobic zone	• Shock loading of readily biodegradable material (VFAs) that promotes high levels of orthophosphate to be released within anaerobic zone • High return rate from solids processing • Long SRT within primary clarifier causing fermentation and high levels of readily biodegradable material (VFAs)
	• No uptake of orthophosphate within the aerobic zone (no decrease in orthophosphate between anaerobic and aerobic zones) • Low DO within aerobic zone	• DO meter, probe, or control system failure (Chapter 13) • Low oxygen transfer due to fouled diffuser, leak in air piping (Chapter 13)
High effluent phosphorus concentration leaving secondary treatment process.	• Increase in orthophosphate prior to and after aerobic zone	• Long HRT within aerobic zone causing release of orthophosphate taken up in biological phosphorus removal process

Table 16-7 – Biological Phosphorus Removal – Common Problems and Potential Causes (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Intermittent fluctuation in phosphorus concentration in recycle line.	• Variable TP and orthophosphate concentration within the RAS line	• Non-continuous operation of sludge wasting leading to variable orthophosphate and TP concentrations returned to biological process • Long SRT within sludge blanket causing release of orthophosphate from settled solids

16.4.4 Optimization of Biological Phosphorus Removal

As noted previously, phosphorus removal is a two-step process in which soluble phosphorus is first converted to a particulate form and the particulate phosphorus is then removed by a solids separation process. Optimization of the solids separation processes are discussed in Chapters 11, 14 and 15. Optimization of biological phosphorus removal processes involves ensuring that the conditions in the anaerobic zone are optimized to store carbon for subsequent usage in the aerobic zone to uptake phosphorus.

Table 16-8 provides common methods to optimize biological phosphorus removal processes.

Table 16-8 – Biological Phosphorus Removal – Optimization of Process

Parameter	Influence on Biological Phosphorus Removal Process	How to Optimize
ORP Within Each Zone	ORP measurements can be used to ensure that the conditions within the anaerobic and aerobic zones are favourable. The ORP for each system varies and therefore comparison relative to the other ORP values in each zone is required. Low (negative) ORP values are expected in the anaerobic zone indicating that no nitrate or oxygen is present. Higher ORP (positive) values are expected within the aerobic zone indicating the presence of dissolved oxygen.	• If high ORP values are present in the anaerobic zone, then attempt to minimize/eliminate the following: air entrainment during mixing, concentration of oxygen and/or NO_3^--N in the return lines, or minimize the impact of high DO concentrations in the influent • Low ORP in the aerobic zone indicates low dissolved oxygen concentrations. Aeration system optimization is presented in Chapter 13

Table 16-8 – Biological Phosphorus Removal – Optimization of Process (continued)

Parameter	Influence on Biological Phosphorus Removal Process	How to Optimize
BOD:TP Ratio Entering the Anaerobic Zone	A BOD:TP ratio of less than 20 indicates low levels of readily available organics, which can decrease the ability of PAOs to uptake/store carbon and release phosphorus in the anaerobic zone. The net effect is a decreased in the overall phosphorus removal efficiency.	• Dosing of acetic acid to the anaerobic zone or installation of a fermentation process to produce VFAs for recycle to the anaerobic zone
SRT	Long SRT within each zone can impact the phosphorus removal process by either causing uptake of orthophosphates in the anaerobic zone or release of orthophosphates in the aerobic zone.	• Increase the sludge wasting rates to decrease the SRT

Optimization of biological phosphorus removal processes to achieve very low effluent TP concentrations will typically require combining biological and chemical phosphorus removal. Care should be taken when combining the two processes to ensure that the biological process is not negatively impacted by chemical addition. More information on chemical phosphorus removal is provided in Section 16.3.

16.5 CAPABILITIES OF PROCESS TECHNOLOGIES FOR PHOSPHORUS REMOVAL

The effluent concentration achievable by a phosphorus removal process depends on two factors: the efficiency of the process to convert soluble phosphorus to particulate phosphorus, and the effectiveness of the solid-liquid separation process for removal of suspended matter.

Figure 16-9 illustrates the impact that solids can have on the effluent concentration of phosphorus. In a phosphorus removal process, the phosphorus content of the suspended solids typically ranges from 3 to 5 percent. Based on a phosphorus content of 3 percent (TP/TSS = 0.03), an effluent containing 10 mg/L of TSS would have a total phosphorus of 0.3 mg/L, excluding the concentration of any remaining soluble phosphorus. For this reason, when low levels of effluent phosphorus are required, tertiary treatment to remove effluent suspended solids is commonly used. Information on tertiary treatment is presented in Chapter 15.

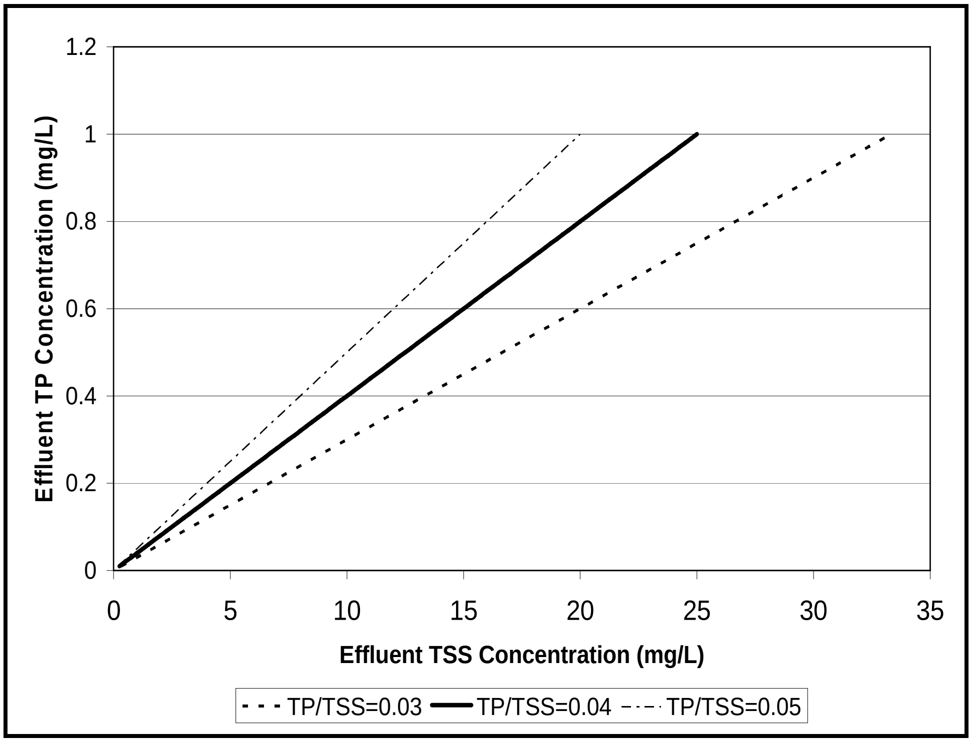


Figure 16-9 –Impact of Effluent Suspended Solids on Effluent Phosphorus Concentration

Table 16-9 presents a number of process and system configurations as well as the average effluent total phosphorus concentration that might be achievable by optimization of that phosphorus removal process configuration. This table does not contain a complete listing of all potential phosphorus removal processes but examples of a wide range of chemical, biological and combined phosphorus removal processes. The information for Table 16-9 was gathered from Nutt (1991) and EPA (2007).

Table 16-9 – System Configuration and Expected Effluent TP Concentrations

Treatment System	Phosphorus Removal Chemical Addition	Expected Effluent Quality (mg/L)	Comments
Conventional Activated Sludge (CAS)	Simultaneous	0.1-0.5[1]	• 1 plant reviewed with capacity of 3,200 m^3/d

Table 16-9 - System Configuration and Expected Effluent TP Concentrations (continued)

Treatment System	Phosphorus Removal Chemical Addition	Expected Effluent Quality (mg/L)	Comments
Conventional Activated Sludge (CAS)	Pre- and post-precipitation	0.2[1]	• Metal/TP weight ratio of 1.5 to 2 required • Addition of polymer and metal salt • Equalization and effluent polishing required
CAS with Filtration	Unknown addition point(s)	0.04[2]	• 1 plant reviewed with capacity of 3,100 m^3/d with unknown filtration system
CAS with Sand Filtration Followed By Microfiltration	Unknown addition point(s)	<0.05[2]	• 1 plant reviewed with capacity of 680 m^3/d
BNR with Filtration	None	0.1 - 0.2[2]	• 1 plant reviewed with a capacity of 19,000 m^3/d
	Pre- and post-precipitation	0.07 [2]	• 1 plant reviewed with a capacity of 91,000 m^3/d
BNR with Tertiary Settling and Filtration	Post-precipitation	0.007-0.065[2]	• 5 plants reviewed with a capacity ranging from 5,700 to 25,000 m^3/d
Extended Aeration (EA) Plus Filtration	Unknown addition point(s)	0.1-0.2[1]	• 9 plants reviewed with capacities ranging from 540 to 12,070 m^3/d
EA Plus Two Stage Sand Filtration	Post-precipitation	<0.011[2]	• 2 plants reviewed with capacities of between 1,890 to 5,870 m^3/d
Oxidation Ditch with Filtration	Unknown addition point(s)	0.058-0.07[2]	• 2 plants reviewed with capacities of between 8,700 and 21,000 m^3/d • Filtration included membrane or multi-media traveling bed filtration

Table 16-9 – System Configuration and Expected Effluent TP Concentrations (continued)

Treatment System	Phosphorus Removal Chemical Addition	Expected Effluent Quality (mg/L)	Comments
Rotating Biological Contactor (RBC), Sand Filters, and Microfiltration	Unknown addition point(s)	<0.04 – 0.06(2)	• 2 plants reviewed both with capacities of 1,900 m^3/d

Notes:

1. Nutt (1991).
2. The average of the monthly average measurements as reported in EPA (2007).

16.6 CASE HISTORIES

16.6.1 Barrie Water Pollution Control Centre (Ontario)

The following case study is based on information presented in Olsen *et al.* (2007).

The City of Barrie undertook a plant optimization and upgrade in order to increase the capacity of the Water Pollution Control Centre (WPCC) from 57,100 m^3/d to 75,000 m^3/d. The treatment processes at the Barrie WPCC include a high purity oxygen activated sludge process followed by secondary clarification, RBCs, four low head gravity filters with silica sand media, and UV disinfection. Dual point alum addition also takes place with dosing between the activated sludge tanks and the secondary clarifier as well as between the RBCs and the filters. A process flow diagram of the existing process is presented in Figure 16-10.

The planned expansion to 75,000 m^3/d would involve the installation of two additional low-head gravity filters and a series of flash and flocculation tanks located downstream of the RBCs to optimize chemical dosing. The City also undertook an optimization study of the dual-point chemical addition system in order to optimize the chemical dosage at each additional location.

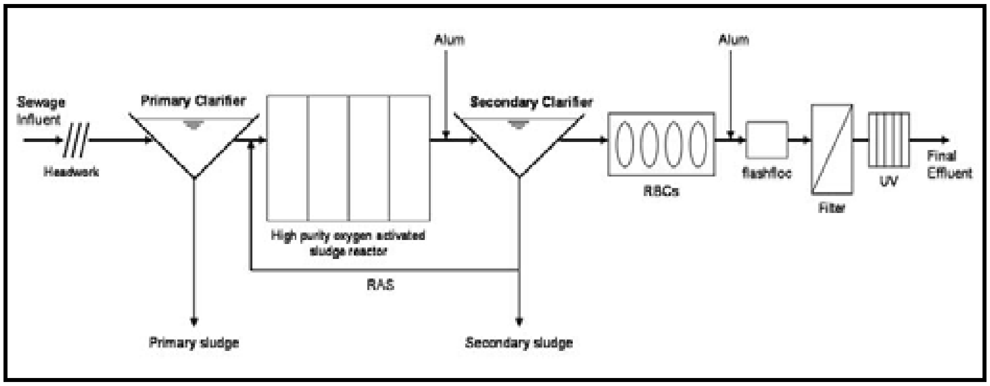


Figure 16-10 – Process Flow Diagram of the Barrie Water Pollution Control Centre

Prior to the design of the plant expansion and upgrade, extensive testing of numerous combinations of chemical addition points took place over the course of a year. It was determined that dual point addition should continue for the following reasons:

- chemical use for P removal is lowered; and
- optimization of chemical addition can enhance the performance of tertiary filters.

It was identified that alum addition between the activated sludge process and the secondary clarifiers and between the RBCs and the newly constructed flashfloc tanks would be the most effective. If required, the option to dose alum at several additional points is also possible including in the activated sludge process and into the flocculation tank downstream of the RBCs.

The optimization study confirmed that the new operating strategy was able to improve the effluent such that it can achieve the plant's new non-compliance limit of 0.18 mg/L-P.

Currently, phosphorus is monitored off-line on a daily basis by sampling out of one of the six UV effluent troughs. In order to be able to more effectively monitor the process, a chemical analyzer is being installed that is capable of monitoring orthophosphate, ammonia, and nitrate. Upon start-up, the system will be operated manually with the option to automate. This could be used in the future to further optimize the chemical dosage.

16.6.2 Rock Creek Advanced Wastewater Treatment Facility (Oregon)

The following case study is based on information presented in Johnson *et al.* (2005).

The Rock Creek Advanced Wastewater Treatment Facility (AWTF) located in Oregon has a maximum month flow of 185,500 m^3/d. The facility uses a combination of CEPT, secondary and tertiary treatment to achieve the required monthly median total effluent phosphorus concentration of 0.07 mg/L or less during the dry season. Figure 16-11 presents a process diagram of the facility. The plant is only obligated to remove nitrogen and phosphorus during the dry season from May 1 to October 31.

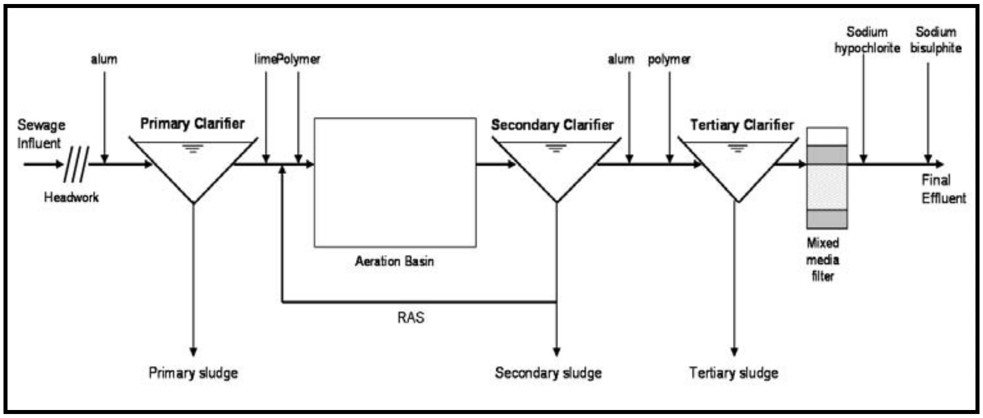


Figure 16-11 - Process Flow Diagram of the Rock Creek Advanced Wastewater Treatment Facility

The plant has two treatment trains (West Side and East Side) which split after preliminary and primary treatment (3 mm screens, primary clarification with alum dosing to enhance phosphorus removal and enhance solids separation). Alum is dosed by continuous flow-pacing at a set point of between 15 and 25 mg alum per litre of primary influent. The alum dosage set point is adjusted manually based on the results of composite primary effluent sampling and analysis. The orthophosphate concentration entering secondary treatment is approximately 1.5 mg/L. This concentration of phosphorus is required in order to ensure that nutrients are not limited within the downstream bioreactors.

The East Side treatment train contains three bioreactors, each followed by a secondary clarifier. Two of the bioreactors have an anoxic zone followed by an aerobic zone (known as a Modified Ludzack-Ettinger (MLE) bioreactor) and the third is a step-feed anoxic zone plus aerobic zone bioreactor. Following secondary clarification, alum and polymer are dosed for soluble phosphorus removal in a solids contact tertiary clarifier (Claricone System). The average alum dosage for the East Side of the Rock Creek AWTF is 45 mg/L. The tertiary effluent is then filtered in a deep bed (1.2 m) filter before chlorine disinfection, dechlorination and discharge.

The West Side treatment train begins with two MLE bioreactors operated in parallel. The flow streams from the bioreactors are then combined before entering four secondary clarifiers. After clarification, the secondary effluent is directed into a rapid mix and flocculation system by vertical turbine mixers, where alum is dosed. Subsequently, the flow is directed to tertiary filters, disinfection, dechlorination and discharge.

From both treatment trains, the primary and secondary solids are combined, thickened on a gravity belt thickener, and then sent to an anaerobic digester. The plant utilizes centrifuges to dewater sludge. The tertiary sludge is combined with the primary influent rather than sent to sludge thickening because the operations staff has found that this decreases the alum dosage required in the primary clarifiers.

As the description of the facility's treatment process above indicates, phosphorus is reduced through a four-step process:

- alum precipitation in the primary clarifiers;
- metabolic phosphorus uptake in the bioreactors due to cell growth (not due to the presence of PAOs);
- alum precipitation within the tertiary clarifier system; and
- filtration to remove particulate phosphorus.

The effluent phosphorus concentrations leaving the tertiary filters versus the molar alum dosage is shown in Figure 16-12 for the East Side. The figure shows that above a molar ratio of 4 to 5, there is no improvement in phosphorus removal.

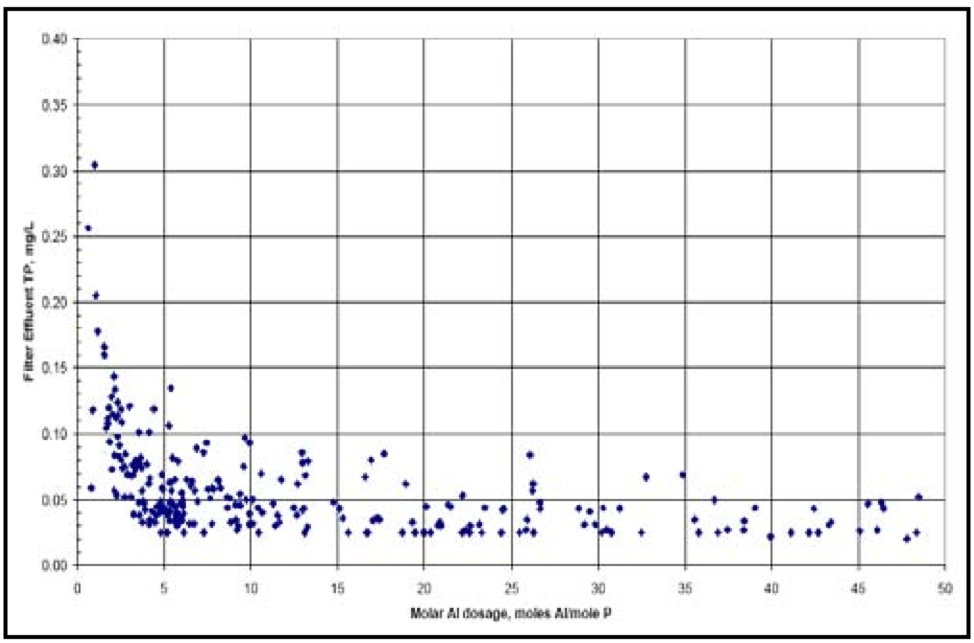


Figure 16-12 – Rock Creek AWTF Phosphorus Removal

16.6.3 Durham Advanced Wastewater Treatment Facility (Oregon)

The following case study is based on information presented in Johnson *et al.* (2005) and EPA (2007).

The Durham AWTF located in Oregon has a capacity of 102,200 m^3/d. The facility uses a combination of biological phosphorus removal (BPR), chemical addition, tertiary clarification and granular media filtration. A schematic of the treatment process is presented in Figure 16-13. Alum is dosed upstream of the tertiary clarifiers to precipitate phosphorus. The facility is required to produce a monthly median total effluent phosphorus concentration of 0.07 mg/L or less during the dry season (May to October). The plant is not required to remove nitrogen or phosphorus from November to April.

The sewage is screened and degritted in a vortex grit separator. The degritted sewage is then split as it flows through the primary clarifiers, recombined and then split again as it flows to the four secondary treatment trains. Each secondary treatment train consists of seven-cell bioreactors that have the flexibility to operate in several different BPR modes. The facility is typically set up in an A^2/O configuration where the RAS and primary effluent are combined in the first zone (cell) followed by two anaerobic zones and two anoxic zones in which nitrified mixed liquor is recycled from the end of the aerobic zones. The last two cells are operated aerobically, as plug flow rather than completely mixed reactors. The mixed liquor then flows to dedicated circular secondary clarifiers before being combined prior to tertiary treatment.

The target orthophosphate concentration is between 1 to 2 mg/L for the secondary effluent. If any of the bioreactors exceed this range, alum is dosed prior to the secondary clarifiers. The facility is typically required to dose alum to the secondary treatment between two to four times per year.

Dosing alum to the BPR process can lead to operational difficulties due to the fine balance between allowing the BPR to function and supplementing with chemicals. Chemical precipitation by alum can impair a BPR process by taking available phosphorus away from the BPR process, leading to negative impacts on the BPR biomass. In addition, as the chemical precipitation sludge produced will continue to be present within the reactor long after dosing is stopped, the impact of alum dosing can be long term.

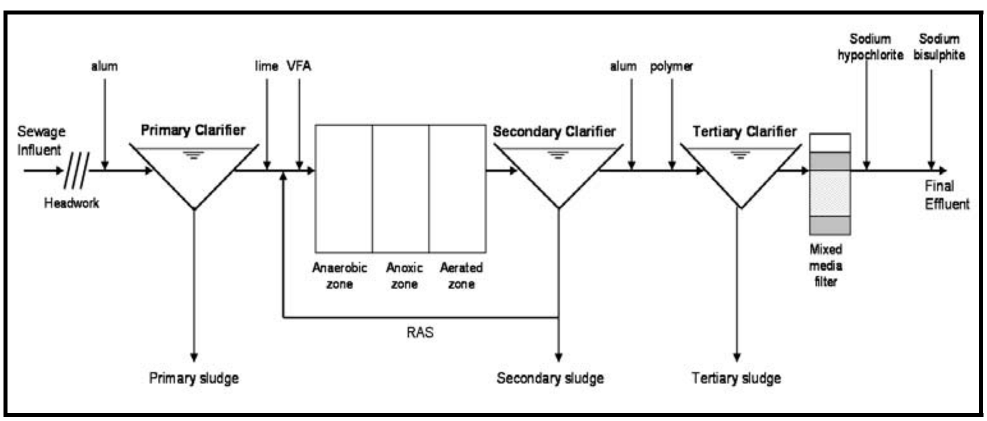


Figure 16-13 – Process Flow Diagram of the Durham Advanced Wastewater Treatment Process

Tertiary treatment includes three two-stage rectangular tertiary clarifiers with "squircle" sludge removal mechanisms. This type of sludge removal process is unusual and, as a result of sludge removal limitations in the corners of the clarifiers, sludge build up as well as re-suspension of solids can occur, leading to a lower than expected solids capture. Each clarifier has two square sections separated by a baffle wall. In the first section of the clarifier, alum is dosed prior to a horizontal paddle flocculator which is separated from the main clarifier by a fabric wall. This flocculator was installed as a retrofit to the original clarifier and has not yet been optimized. The average alum dosing for the Durham AWTF is approximately 45 mg/L. For the most part, sludge separates out in the first section prior to flowing past the baffle wall into the second section. From the tertiary filters, the effluent is disinfected in a chlorine contact chamber prior to entering the conventional multi-media (four layer) filters. The effluent is dechlorinated prior to discharge.

The solids processing at the Durham AWTF includes thickening of the primary sludge and WAS separately before mesophilic anaerobic digestion. The facility has developed a patented fermentation system for primary sludge that is used to increase the VFAs within the secondary treatment bioreactors. Fermentation takes place in the first stage of the thickener and then both the overflow and thickened sludge are recombined and sent to a second gravity thickener for further thickening. The VFA rich overflow from the second thickener is then returned and mixed with the primary effluent. The thickened sludge is sent to be anaerobically digested.

The Durham AWTF utilizes a three-step process to remove phosphorus:

- Biological phosphorus removal through a seven-celled bioreactor operated in an A^2/O configuration;
- Alum dosing in the tertiary clarifier prior to horizontal flocculators; and
- Provision for alum dosing to the secondary treatment system in case the BPR is not providing the required phosphorus removal.

Figure 16-14 presents the effluent phosphorus concentrations leaving the tertiary filters versus the molar alum dosage for the Durham AWTF.

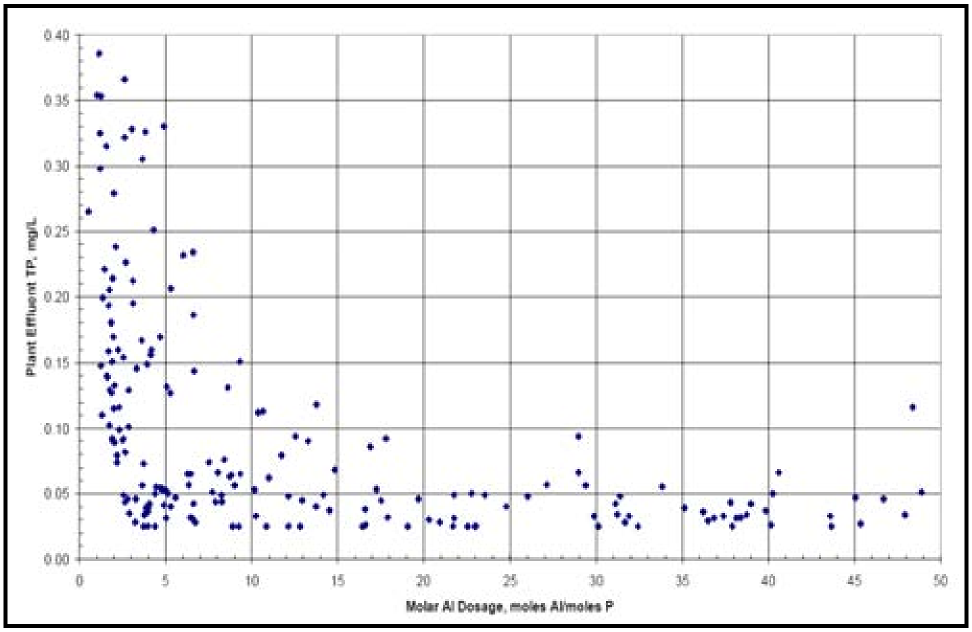


Figure 16-14 – Durham AWTF Phosphorus Removal

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CHAPTER 17

LAGOON-BASED SYSTEMS

CHAPTER 17

LAGOON-BASED SYSTEMS

17.1 OVERVIEW OF LAGOON-BASED SYSTEMS

17.1.1 Types of Lagoon-Based Systems

Lagoon-based sewage treatment systems are commonly used in smaller communities in Ontario. There are a number of different types of lagoons and many possible system designs. Some lagoon designs provide adequate treatment alone and others are used in combination with additional treatment processes. The terms "lagoon" and "pond" are often used interchangeably, and names such as "polishing", "stabilization" and "maturation" can refer to a lagoon's treatment role.

In Ontario, lagoons can have either a continuous discharge or a controlled discharge (also known as "fill and draw"). Effluent is released seasonally for controlled discharge systems, typically with no discharge during the winter months when the lagoons may be ice-covered, or during the summer when the assimilative capacity of the receiving stream may be limited. However, due to certain site specific circumstances, winter discharge may be required.

There are four main types of lagoons based on the environment that is maintained in the lagoon, namely:

- aerated lagoons;
- aerobic lagoons;
- facultative lagoons; and
- anaerobic lagoons.

Table 17-1 provides a brief description of each lagoon type.

Table 17-1 - Types of Lagoon-Based Systems

Type of System	Description
Aerated	Use mechanical aerators or diffused aeration systems to mix lagoon contents and add oxygen. Can be either complete-mix or partial-mix lagoons, depending on the extent of aeration.
Aerobic	Not mechanically aerated, but dissolved oxygen is present throughout much of the lagoon depth, from algae photosynthesis and surface diffusion enhanced by wind, as this lagoon type is usually shallow.
Facultative	Aerobic, anoxic and anaerobic conditions typically exist in the lagoon, although it may be aerobic through the entire depth if lightly loaded. Also commonly known as “stabilization ponds” or “oxidation ponds”. No mechanical aeration is provided and due to depth of the lagoons, there is stratification of the DO concentration, with the top being aerobic, middle anoxic and bottom anaerobic. The bottom anaerobic zone contains settled sludge.
Anaerobic	Typically used to treat animal waste or other high strength wastewater, or as a first treatment stage in systems using two or more lagoons in series. Dissolved oxygen is largely absent from the lagoon and treatment is provided by anaerobic bacteria. This type of lagoon is seldom used in Ontario for municipal wastewater treatment and is not discussed further in the Guidance Manual.

17.1.2 Evaluating Process Performance

The effectiveness of lagoon-based systems is measured by the ability to reduce BOD_5 and TSS to meet the effluent requirements for the plant discharge. Typical lagoon effluent $CBOD_5$ values range from 20 mg/L to 60 mg/L, and TSS levels will usually range from 30 mg/L to 150 mg/L. Nitrification may be provided by some systems, and the effectiveness of nitrifying lagoons is determined from the analysis of effluent TAN concentration, and is influenced greatly by temperature and mixing conditions. Removal of TP may be provided by chemical dosing with chemical coagulants. For plants using chemical dosing for TP removal, lagoon effluent TP concentrations of < 1 mg/L can typically be achieved.

In addition to influent and effluent quality data assessment, the influent and effluent flow, liquid level and sludge volume can be monitored as an indicator of process performance. Microscopic evaluation of the lagoon can also be used as an evaluation tool. Monitoring and recording these parameters over time can be used to identify potential problems with a lagoon’s operation.

Table 17-2 presents monitoring recommendations, in terms of sampling locations and analyses, in order to evaluate the performance and efficiency of lagoon-based systems. The evaluation of performance can be based on the plant’s ability to achieve its regulatory requirements as outlined in the C of A for the treatment facility.

BOD_5, TSS and TP sampling should be conducted at least weekly, and may be required daily for the final effluent when lagoons are discharging. For nitrifying lagoons, weekly sampling for influent and effluent pH, TKN and effluent TAN should also be carried out. Sampling should be carried out between lagoons when there is more than one lagoon cell operating in series.

It is recommended that liquid level measurement be conducted at least monthly to obtain representative operational and performance data in controlled discharge lagoons where liquid level can vary. For continuous discharge lagoons, influent and effluent flow data can be compared and used to determine if exfiltration or infiltration is occurring.

It is recommended that the depth of sludge be monitored annually, unless the lagoon receives a high level of solids (e.g. waste sludge from clarifiers), in which case more frequent monitoring is recommended. If on-line flow monitoring is available, data should be collected and trended over a period of time to assess the impact of seasonal flow variations on the lagoon treatment process.

A microscopic evaluation of lagoon effluent can be a useful tool for monitoring the health of the lagoon ecosystem. The presence or absence of certain microorganisms is an indicator of potential problems, e.g. high sulphur bacteria.

Tracer testing can be used to identify any short-circuiting in the lagoon cells. Refer to Section 14.4.1 for details on tracer testing. It should be noted that due to the size of the lagoon cells, which provide retention times on the order of days, the procedures outlined in Section 14.4.1 may have to be altered.

Table 17-2 – Recommended Process Monitoring to Evaluate Performance

Location	Parameter	Comments
Influent Flow	On-line total daily flow metering	Used to determine the hydraulic retention time in the lagoon.
Effluent Flow	On-line total daily flow metering	Can be used to determine if exfiltration or infiltration is occurring in continuous discharge lagoons. Also likely part of a C of A for controlled and continuous discharge lagoons.
Influent and Effluent	BOD_5, TSS	Used to evaluate effectiveness of treatment. Composite sampling recommended. May be required for effluent monitoring as part of the C of A.
Influent and Effluent	pH, TKN, TAN, NO^-_2-N, NO^-_3-N	Used to evaluate nitrification for nitrifying lagoons. Composite sampling recommended.

Table 17-2 - Recommended Process Monitoring to Evaluate Performance (continued)

Location	Parameter	Comments
Influent and Effluent	TP	Evaluate effectiveness of chemical addition for TP removal.
Lagoon	Liquid level(1)	Data gathered over longer time periods can be used as a reference when water levels vary significantly.
Lagoon	Sludge level	Data gathered over longer time periods can be used as a reference if sludge levels suddenly change. Used to determine when desludging is required.
Lagoon	DO profile, temperature profile	Can identify areas where treatment may be affected by variations in temperature or DO concentration.
Effluent	Microscopic examination	Can identify and quantify specific microorganisms that indicate problems with the lagoon's operation.
Note: 1. For controlled discharge lagoons.		

17.1.3 Common Problems and Potential Process Impacts

Changes in lagoon treatment performance can result from changes in loading, DO levels, temperature, sunlight or other influences. In terms of the system design, HRT and lagoon depth have a significant impact on performance as do factors that affect these parameters (e.g. short-circuiting, excessive sludge accumulation). All of these factors can affect the microbial ecosystem in the lagoon which can alter the lagoon water chemistry.

The factors that can impact the process performance of a lagoon-based system are presented in Table 17-3. Table 17-4 presents the symptoms and causes of common problems encountered with lagoon-based systems.

Table 17-3 – Factors Affecting Process Performance of Lagoon-Based Systems

Factor	Potential Process Impacts
Light	• Low light levels (e.g. from ice and snow cover) will result in lower photosynthesis by algae and other photosynthesizing organisms • Lower photosynthesis can reduce DO level in the lagoon • Lower photosynthesis can affect nutrient (N and P) removal • Rate of surface disinfection is proportionate to the amount of sunlight, so lower light levels can result in higher effluent coliform counts
Temperature	• Lower temperatures can result in lower BOD_5 and TAN removal rates due to lower growth rates of lagoon microorganisms • For facultative lagoons, vertical stratification of temperature can occur during certain times of the year, which can result in a phenomenon known as “overturn” in fall and spring. Overturn events can lead to odour releases from lagoons • Increases in sludge decomposition rates during the onset of warmer temperatures in the spring can negatively affect effluent quality through the release of soluble organics and nutrients • Short-circuiting can occur during colder months in facultative lagoons as warmer wastewater can flow across the surface of lagoon without mixing with colder lagoon water • Although not designed for this purpose, lagoons may experience ammonia removal by volatilization that naturally occurs at warm temperatures when accompanied by high pH conditions in aerated and facultative lagoons
Sludge depth	• An accumulation of sludge can reduce available water depth that will reduce the effective HRT, which can negatively affect treatment performance • Excess sludge accumulation levels may result in odour problems • Excessive sludge accumulation can affect effluent quality as organic matter is solubilized and nitrogen and phosphorus can be released from the anaerobic sludge
Loading	• Loads in excess of the lagoon’s treatment capacity can produce poor quality effluent. In some cases, odour problems may occur

Table 17-4 - Lagoon-Based Systems - Symptoms and Causes of Common Problems

Problem	Common Symptoms	Common Causes
Poor BOD_5 removal.	• Elevated $CBOD_5$ levels in effluent	• Low DO • Low temperature • Organic overloading • Short HRT, either due to high influent flow or short-circuiting • Higher algae growth • High sulphur bacteria growth • Floating sludge
Poor TSS removal.	• Elevated TSS levels in effluent	• Organic overloading • Excessive algae growth • High sulphur bacteria growth • Floating sludge • Excessive mixing and resurfacing of solids due to wind effects
Poor nitrification.	• Elevated TAN levels in effluent • Build up of nitrite in effluent (1- 2mg/L)	• Low temperature • Low DO and/or low pH • Organic overloading • Low alkalinity • Release of TAN from accumulated sludge
Low effluent pH.	• pH <7	• Organic overloading • Low DO • Low alkalinity in nitrifying lagoons
High effluent pH.	• pH >9	• Excessive algae growth
Odour.	• Persistent odour from part or all of lagoon	• Organic overloading • Aerator problems (for aerated lagoons) • Excessive sludge accumulation • Result of spring melt if ice-covered in the winter • Overturn of layers with changes in temperature for facultative lagoons (typically happens in spring or fall)
Excessive sludge accumulation.	• High sludge levels in part of lagoon • Poor effluent quality as a result of lower HRT, sludge decomposition by-products and/or loss of old sludge in effluent	• Too long a time interval between sludge removal • Short-circuiting
Excessive algae growth.	• Elevated effluent TSS levels	• Long HRT • Excessive mixing • Release of nutrients from excessive sludge accumulation

17.2 OPTIMIZATION APPROACHES

Lagoon optimization depends on the correct diagnosis of the problem and understanding the ecology of the lagoon system. A number of options exist to optimize the treatment performance of a lagoon-based system, but some problem factors (e.g. organic overloading and low temperature) may not be correctable through optimization alone.

Table 17-5 presents a summary of potential optimization solutions for common problems with lagoon-based systems. It should be noted that some potential solutions can have an impact on other areas of lagoon performance, and the potential impact of proposed changes should be considered as part of an optimization program. The operational changes that can be made to a lagoon system are limited; therefore, in order to optimize performance, the process would need to be changed to some degree.

Table 17-5 – Optimizing Lagoon-Based Systems

Potential Problem	Potential Symptom	Possible Solutions
Low DO.	• High effluent BOD_5 • High effluent TAN (for nitrifying lagoons) • Low effluent pH	• Check that aerators are operating correctly for aerated lagoons, and if not, correct the problem • Consider installing additional aeration or alternative oxygenation supply capacity • Convert facultative lagoon to aerated lagoon • Install additional lagoons for pretreatment or polishing • Consider installing supplemental oxygenation
Organic overloading.	• Low DO in the lagoon • High effluent BOD_5 • High effluent TSS • High effluent TAN (for nitrifying lagoons) • Low effluent pH • Odour	• Check that aerators are operating correctly for aerated lagoons, and if not, correct the problem • Consider installing additional aeration or alternative oxygenation supply capacity • Convert facultative lagoon to aerated lagoon • Install additional lagoons for pretreatment or polishing • Consider installing supplemental oxygenation
Short-circuiting.	• High effluent BOD_5 • High effluent TSS • High effluent TAN (for nitrifying lagoons) • Excessive sludge accumulation	• Desludge if sludge accumulation is excessive • Install curtain wall or berm in the lagoon to provide better separation of lagoon influent and effluent points • Convert facultative lagoon to aerated lagoon (mixing will prevent short-circuiting in winter that is due to influent and lagoon temperature differences)

Table 17-5 – Optimizing Lagoon-Based Systems (continued)

Potential Problem	Potential Symptom	Potential Solutions
Excessive algae growth.	• High effluent TSS • High effluent pH (> 9) • Increased algae growth in lagoon and increase in algal cells in effluent	• Reduce the HRT by dividing lagoons with curtain walls or berms • Check the level of mixing in aerated lagoons if applicable and optimize to provide some turbidity (inhibits light penetration), but not too high to disturb settled sludge (releases nutrients that can promote algal blooms) • Desludge if sludge accumulation is excessive, as sludge can decompose and release nutrients that can promote excessive algal growth • Cover part of the lagoon with a floating cover or fabric to block out sunlight • Add chemicals, such as chlorine and/or copper sulphate. Care must be taken to ensure a non-toxic effluent is discharged if this option is used • Add a commercially-available non-toxic dye that blocks out specific light rays that algae need for photosynthesis
High sulphur bacteria growth.	• Low DO in lagoon • High effluent BOD_5 • High effluent TSS • Sulphur bacteria in effluent	• These indicate an anaerobic environment in the lagoon. Aeration should be checked for aerated lagoons • Check organic loading, and if exceeds design capacity consider installing additional lagoon capacity, or load reduction if an industrial source
Excessive filamentous bacteria.	• Low DO in lagoon • High effluent BOD_5 • High effluent TSS	• These indicate a low DO environment in the lagoon. Aeration should be checked for aerated lagoons • Check organic loading, and if exceeds design capacity consider installing additional lagoon capacity, or converting facultative lagoon to aerated lagoon
Low alkalinity.	• Higher effluent TAN levels • Effluent nitrite > 1 mg/L	• Add lime or other source of alkalinity to the lagoon during nitrification periods
High effluent pH.	• Excessive algae growth	• Reduce the HRT by dividing lagoons using curtain walls or berms or other method • Check the level of mixing in aerated lagoons. Mixing should sufficient to create some turbidity to block sunlight, but not too high as to disturb settled sludge • Desludge if sludge accumulation is excessive as sludge can decompose and release nutrients that can promote excessive algal growth

Table 17-5 – Optimizing Lagoon-Based Systems (continued)

Potential Problem	Potential Symptom	Potential Solutions
Excessive sludge accumulation.	• High sludge level in part or all of lagoon • High effluent BOD_5 • High effluent TSS • Excessive algae growth • Odour	• Desludge lagoon

17.3 UPGRADE OPTIONS

In some cases, optimization of a lagoon-based system will not be sufficient to meet effluent discharge quality requirements. In these cases, upgrading a lagoon-based system may be required. The optimum upgrade approach is dependent on the type of lagoon, effluent quality requirements, site conditions, and flow conditions. The cost of an upgrade is an important factor and this must also be taken into account when determining the most feasible option. Typically, upgrading a lagoon system will be the lower cost option compared to replacing lagoons with a mechanical sewage treatment plant. The following subsections discuss upgrade options for lagoon-based systems.

17.3.1 Upgrading Aeration Systems

Facultative lagoons can be upgraded to aerated lagoons to improve the treatment performance. This will typically increase BOD_5 removal as a result of higher oxygenation rates and DO levels in the lagoon and the elimination of ice and snow cover in the winter. It may also allow for ammonia reduction in the lagoon during warmer months.

Options for upgrading facultative lagoons to aerated lagoons can either involve installing surface aerators or using a diffused air system. When determining the best option for aeration, the aeration requirements, mixing requirements and energy use for each option should be considered. In addition, ease of installation and capital and O&M costs need to be taken into account.

17.3.2 Additional Lagoon Capacity

In cases where the HRT is too low or the organic loading exceeds the design capacity for a lagoon-based system, installing an additional lagoon can be used to upgrade the treatment system. The additional lagoon(s) can be designed to pre-treat wastewater prior to entering the existing lagoon, or as a final lagoon to polish the effluent. In some cases, improved treatment performance can be provided by a lagoon system with two or more lagoons operating in parallel and reconfiguring these to a series operation. Alternatively, additional lagoon capacity can be provided by adding one or more lagoons to operate in parallel with the existing lagoon(s).

Adding additional lagoon capacity requires sufficient land available for constructing new lagoon cell(s). This option may not be suitable for facilities that need to upgrade to provide nitrification during colder months as low temperatures (< 10 ^{o}C) inhibit the nitrification process.

17.3.3 Installation of an Insulated Cover

Low temperatures can have adverse affects on the performance of lagoon systems. The installation of an insulated cover over aerated lagoons can serve to minimize the effects of low ambient temperatures, including freezing and ice build up on mechanical aeration equipment, on lagoon performance. The insulated cover can also minimize ice build up on the lagoon surface and at the lagoon discharge, which can affect the ability to discharge during winter months, if required.

17.3.4 Additional Options

Additional treatment processes can be installed at a lagoon-based treatment system to improve the treatment performance and achieve secondary effluent quality. The type of upgrade is dependent on the effluent quality requirements, as discussed below.

Intermittent sand filters (ISF) are an upgrade option for lagoon-based systems. Passing lagoon effluent through an ISF process can reduce lagoon effluent TSS, TP and TAN. Lagoon effluent is either pumped or fed by gravity over the surface of the filter. Particulate matter (and associated BOD_5 and nutrients) is removed as lagoon effluent flows through the filter. Nitrification can also occur as nitrifying microorganisms can attach to or grow on the filter media. Filter backwashing is not required. The filter is taken off-line for cleaning or self-regeneration when the filter surface becomes plugged. Use of ISF technology may also provide additional treatment capacity for controlled discharge lagoon systems by allowing continuous discharge or an increase in the duration of the discharge period.

ISF has been used at a number of facilities for polishing of lagoon effluent. One of the earliest demonstrations of the ISF process to treat lagoon effluent in Ontario was in New Hamburg. Since the installation of the ISF at New Hamburg in 1980, a number of other facilities have installed an ISF system to treat lagoon effluent in Ontario, including Norwich, Lakefield and Exeter. An example of typical effluent quality for lagoon treatment followed by ISF is presented in Table 17-6.

Table 17-6 – Effluent Quality[1] (mg/L) for Norwich WWTP – Lagoon and ISF Treatment

BOD_5	TSS	TP	TAN
5.5	4.6	0.15	0.75 to 2.6

Note:

1. Data for 2005 to 2007. More information on this facility can be found at http://www.county.oxford.on.ca/site/841/default.aspx

Passing lagoon effluent through a settling basin, clarifier, DAF process, microscreen or granular media filter can be used to reduce the level of TSS in the final effluent. In conjunction with chemical addition (e.g. alum), these options can also be used to reduce the final effluent TP concentration. Rapid sand filters and other tertiary filters have been used at some facilities with some success for TSS and TP removal. However, these systems typically do not remove algae as well as ISF. Rapid sand filters require regular backwashing, with the backwash water going back to the lagoon, which can exacerbate any existing problems with excessive algae growth. This approach has been successfully used at Drayton (continuous backwash filters) and Niagara-on-the-Lake (tertiary clarifiers), among others. Constructed wetlands are another option available for improving lagoon effluent quality.

An option to provide nitrification of lagoon effluent is to install a process that provides sufficient surface area and contact time for nitrifying microorganisms, such as a MBBR or other submerged media biological reactor. If nitrification is only required during warmer months installing media in the aerated lagoon that allows the attachment of nitrifying bacteria may be a practical option.

17.4 CASE HISTORIES

17.4.1 Upgrading a Facultative Lagoon to an Aerated Lagoon, Town of Ponoka, Alberta

The following case study is based on information presented in Nelson Environmental Inc. (2002).

The Town of Ponoka, Alberta had a treatment system consisting of anaerobic primary lagoons followed by a facultative lagoon and three storage lagoons. The facility, which was designed to discharge once a year, needed to discharge twice per year as a result of population growth. In order to ensure regulatory compliance, the system would either need to significantly increase storage capacity or an alternative upgrade option was needed.

Upgrade options considered included replacing lagoon treatment with RBCs, an activated sludge process, and a modified SBR. The conversion of the facultative lagoon to an aerated lagoon was also reviewed. A review of these upgrade options identified converting the existing facultative lagoon to an aerated lagoon to achieve better treatment performance as the best option. An aerated lagoon had the lowest combined capital and O&M costs over a 20-year period. This option also allowed for twice yearly discharge due to the effluent quality that could be achieved with an aerated lagoon.

The facultative lagoon has an area of 8.5 ha and a depth of 1.5 m. The lagoon was divided into three cells, operating in series, and fine bubble aeration was installed in each cell. The three-cell design maximizes the treatment performance of the lagoon. The total HRT in the aerated lagoon is approximately 28 days at the design flow of 4,250 m^3/d. The resulting average effluent quality from the new aerated lagoons is 16 mg/L $CBOD_5$.

More information on this upgrade can be found at http://www.nelson environmental.com/casestudies.asp (Nelson Environmental, 2002).

17.4.2 Installation of a Moving Bed Biofilm Reactor to Provide Year-Round Nitrification, Johnstown, Colorado

The following case study is based on information presented in Wessman and Johnson (2006).

An increase in population and the requirement to improve effluent ammonia limits resulted in the Town of Johnstown assessing options to upgrade their wastewater treatment facility. The original plant consisted of three lagoons operating in series. Upgrade options that were considered included replacing the lagoon-based system with an activated sludge plant and maintaining the lagoons and adding a fixed-film treatment process to reduce the lagoon effluent ammonia concentration. After a review of options, the Town decided to install an MBBR process downstream of the lagoons. An MBBR consists of specific plastic media designed to provide a large surface area for biofilm growth. The media is contained inside an aerated tank where sufficient air is added for the microorganisms and also to provide continuous movement of the media. This option was chosen as it has been proven to provide nitrification during colder temperatures and it was also a lower cost option than replacing the lagoons with an activated sludge plant.

The MBBR system was designed and built to treat the current flow of 2,840 m^3/d, whilst accommodating to treatment of the future design flow of 5,680 m^3/d. This was done by sizing the tankage for the future design flow and filling the tank with sufficient MBBR media to treat the current flow. More media will be added with increasing flows as required in the future.

Effluent from the second lagoon flows by gravity to the two-train MBBR process, each train consisting of two basins. The total footprint of the MBBR system is 18.3 m x 18.3 m, and the total volume is 1,223 m^3. MBBR effluent flows to the third lagoon. A new DAF unit was installed after the third lagoon to improve TSS and TP removal. Figure 17-1 provides a process flow schematic of the treatment process.

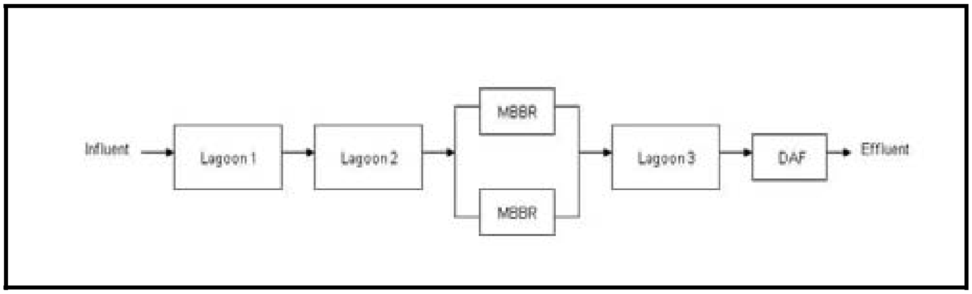

Figure 17-1 – Process Flow Schematic of the Johnstown Wastewater Treatment Plant Process

(Adapted from Wessman and Johnson, 2006)

The MBBR process has achieved the effluent ammonia limits, which are provided in Table 17-7. The final effluent BOD_5 (after DAF treatment) has been less than 1 mg/L, on average.

An MBBR plant was also installed at the wastewater treatment facility in Ste-Julie, Quebec to provide year-round nitrification. Effluent from four lagoons operating in series is pumped through an MBBR, which was installed in 2007. The MBBR process reduced lagoon effluent ammonia from 16.2 mg/L to 3.9 mg/L, on average. Further information on the upgrade of the Ste-Julie facility can be found at http://www.johnmeunier.com/en/vw4/detail2/.

Table 17-7 – Johnstown MBBR Design Temperature and Ammonia Effluent Limits

(Adapted from Wessman and Johnson, 2006)

Month	MBBR Influent Temperature (^oC)	Ammonia Effluent Limit (mg/L)
January	4.5	16
February	5.8	12
March	4.9	5.1
April	9.6	2.9
May	14.1	1.5
June	19	1.3
July	22.3	1.2
August	21.2	1.2
September	16.6	1.1

Table 17-7 – Johnstown MBBR Design Temperature and Ammonia Effluent Limits (continued)

(Adapted from Wessman and Johnson, 2006)

Month	MBBR Influent Temperature (^oC)	Ammonia Effluent Limit (mg/L)
October	11.2	1.1
November	4.9	2.1
December	6.2	13

17.4.3 Use of a Post-Lagoon Submerged Attached Growth Reactor for Cold Weather Nitrification, Steinbach, Manitoba

The following case study is based on information presented in Hildebrand *et al.* (2009).

A submerged attached growth reactor (SAGR) pilot plant has been operating downstream of an aerated lagoon at the Steinbach wastewater treatment facility in Manitoba since 2007. The SAGR was installed to demonstrate nitrification at the facility, where lagoon effluent temperatures can be below 0.5 degrees Celsius.

The single-cell aerated lagoon feeding the SAGR process has an HRT of 28 days. The lagoon effluent flows into two parallel SAGR treatment units. Each SAGR units consist of two aerated gravel beds in series, receiving approximately 23 m^3/d of effluent flow. Mulch was placed over the top of the SAGR beds to insulate the inter-cell piping and to prevent ice formation.

The SAGR effluent ammonia, TSS and $CBOD_5$ levels have been below detection levels for most of the winter, with a maximum monthly average ammonia concentration of 0.61 mg/L and maximum daily average ammonia concentration of 1.63 mg/L. During the winter, lagoon effluent $CBOD_5$ levels can exceed 30 mg/L, but this high level has not inhibited the nitrification process in the SAGR. Figure 17-2 presents a process schematic of the wastewater treatment demonstration system at Steinbach.

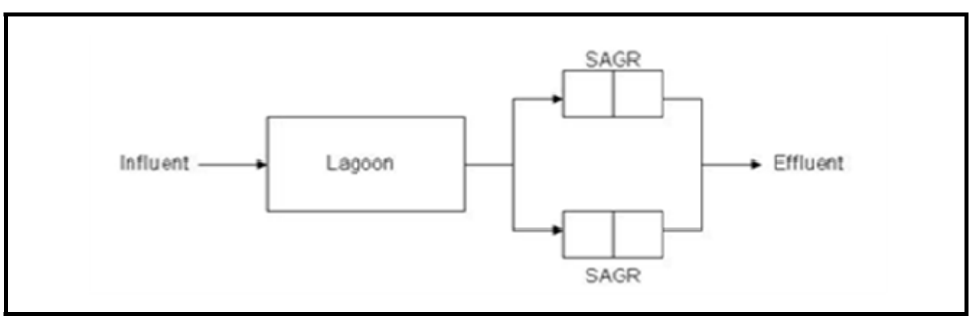

Figure 17-2 - Process Flow Schematic of the Steinbach Demonstration SAGR

(Adapted from Hildebrand *et al.*, 2009)

17.5 REFERENCES

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CHAPTER 18

DISINFECTION

CHAPTER 18

DISINFECTION

18.1 OVERVIEW

Disinfection of sewage treatment plant effluent is required to reduce the quantity of disease-causing organisms discharged into a receiving water body. There are several means to accomplish disinfection of sewage effluent, most commonly accomplished by the use of chemical agents, physical agents, mechanical means, and irradiation.

Disinfection technologies include chemical disinfection with chlorine and its compounds, ozone, bromine, iodine, hydrogen peroxide, and peracetic acid, and other means such UV irradiation, gamma radiation, and ultrasonic irradiation.

This chapter focuses on chlorination / dechlorination and UV irradiation, which are the most commonly used sewage disinfection technologies in Ontario.

18.2 CHLORINATION / DECHLORINATION

18.2.1 Purpose and Chemicals Commonly Used

The most commonly used disinfectant in wastewater treatment in Ontario and many parts of the world is chlorine. Chlorine is a strong oxidant that is highly effective for inactivating bacteria and viruses. It has been found to affect reproduction and metabolism, cause mutations, and ultimately result in death of microorganisms.

The most common chlorine chemicals used for sewage disinfection are chlorine gas (Cl_2), sodium hypochlorite (NaOCl), and calcium hypochlorite [$Ca(OCl)_2$].

Disinfection is achieved following the formation of free and combined chlorine. Free chlorine in the form of hypochlorous acid (HOCl) and hypochlorite ion (OCl^-), are either added or formed by chlorination chemicals. If ammonia is present in the sewage effuent, HOCl will react readily with ammonia to form three species of chloramines, which together are referred to as “combined chloramines” (also referred to as combined chlorine). Combined chloramines are disinfectants; however, they react slower than free chlorine.

Disinfection is a time-dependent process and adequate contact time, typically 30 minutes at average dry weather flow and 15 minutes at peak flow, is required for effective disinfection. In Ontario, disinfection processes are generally designed to meet an effluent *E. coli* objective, which is a typical C of A objective. The required chlorine dosage to achieve a given level of disinfection is a function of the degree of upstream treatment provided. Table 18-1 presents the typical ranges of chlorine dosages required for disinfection of various qualities of effluent and raw sewage.

Table 18-1 – Typical Chlorine Dosages

Operating Condition	Dosage ($mg\ Cl_2/L$)[1]
Raw Sewage	6 - 12 (fresh) 12 - 25 (septic)
Primary Effluent	3 - 20
Trickling Filter Process Effluent	3 - 12
Activated Sludge Process Effluent	2 - 9
Nitrified Effluent	1 - 6
Tertiary Filtered Effluent	1 - 6

Notes:

1. MOE (2008).

Chlorinated wastewater effluent and inorganic chloramines have been declared toxic under the *Canadian Environment Protection Act* (CEPA). Dechlorination of sewage effluent may be required to meet site-specific effluent quality criteria set by MOE, or to meet Pollution Prevention Plan (PPP) requirements under the CEPA, to eliminate chlorine residual toxicity. The most common dechlorinating agents are sulphur dioxide and sodium bisulphite; however, other dechlorination chemicals are also available, some of which are listed in Table 18-2.

The dosage of the dechlorination chemical is a function of the residual chlorine concentration in the effluent. Table 18-2 presents the stoichiometric dosage required to neutralize 1 mg/L total residual chlorine (TRC) for several dechlorinating chemicals.

Table 18-2 – Typical Dechlorination Dosages

Operating Condition	Dosage [1] (mg/mg Cl_2 residual)	Delivered Concentration	Chemical Formula
Sodium Bisulphite	1.46	38% [2]	$NaHSO_3$
Sulphur Dioxide	0.90	99.9% [3]	SO_2
Sodium Sulphite	1.78	96% [4]	Na_2SO_3
Sodium Metabisulphite	1.34	97% [4]	$Na_2S_2O_5$
Sodium Thiosulphate	0.56 @ pH 11	97% [4]	$Na_2S_2O_3$
Calcium Thiosulphate	0.53	30% [2]	CaS_2O_3
Ascorbic Acid	2.5	99% [4]	$C_6H_8O_6$
Hydrogen Peroxide	0.49	35% - 50% [2]	H_2O_2

Notes:

1. Dosages presented are based on stoichiometric requirements for the pure dechlorinating agent.
2. Typically delivered as a liquid solution.
3. Typically delivered as a gas.
4. Typically delivered as a solid.

18.2.2 Evaluating Process Performance

The effectiveness of chlorine disinfection is measured by the ability to meet the requirements for *E. coli* in the plant sewage effluent discharge. Typical *E. coli* effluent objectives and limits in Cs of A issued in Ontario generally range from 100 cfu / 100 mL to 200 cfu / 100 mL based on a monthly geometric mean density.

The TRC is an important benchmark used to control the chemical dosage and ensure adequate disinfection. To achieve the effluent *E. coli* limit, a TRC of 0.5 mg/L after 30 minutes contact time is generally required for effective disinfection of sewage treatment plant effluent (MOE, 2008).

Dechlorination of sewage effluent may be required to meet site-specific effluent requirements for chlorine residual toxicity. The current (2004) CEPA pollution prevention plans specify an effluent TRC limit of 0.02 mg/L, which is essentially a non-detectable level of chlorine. The requirement to produce a non-toxic discharge with respect to chlorine is typically monitored by measuring the TRC or by measuring a slight sulphite residual after dechlorination.

Table 18-3 summarizes the typical benchmarks for the chlorination/ dechlorination process.

Table 18-3 – Typical Benchmarks for Chlorination / Dechlorination

Parameter	Benchmark
Effluent *E. coli* (based on monthly geometric mean)	100 cfu / 100 mL (or as per C of A) (1)
TRC Prior to Dechlorination	0.5 mg/L (2)
Effluent TRC After Dechlorination	<0.02 mg/L (1,3)
Dechlorinating Agent	Detectable

Notes:

1. Typical effluent objectives are 100 cfu/100 mL in Cs of A issued in Ontario.
2. MOE (2008).
3. CEPA requirement.

Table 18-4 presents monitoring recommendations, in terms of sampling locations and analyses, in order to evaluate the performance and efficiency of the chlorination / dechlorination process. The evaluation of performance can be based on the STP's ability to achieve its regulatory requirements in terms of both effluent *E. coli* densities and effluent toxicity (e.g. non-detectable TRC and/or non-acute lethality). The efficiency of chlorination / dechlorination can be evaluated based on the chemical dosages required to meet the regulatory requirements.

TRC sampling should be conducted at least daily, and *E. coli* sampling should be conducted at least weekly over at least a one-month period to obtain representative operational and performance data. If on-line monitoring is available, data should be collected and trended over a period of time to assess the impact of diurnal and peak flows on the disinfection process.

Table 18-4 - Chlorination / Dechlorination – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
STP's Primary Flow Meter	Continuous	• Flow	• Used to estimate the available contact time in the chlorine contact chamber and/or outfall pipe
Secondary/Tertiary Effluent	Grab	• *E. coli*	• Used to assess the log removal of *E.coli* through disinfection
Front End of First Pass of Contact Chamber (immediately after rapid mixing)	On-line / Grab	• TRC	• If the effluent pipe is used for contact time, TRC should be measured at the point of chlorination. Sample should be held for theoretical detention times prior to measuring TRC
Final Effluent	On-line / Grab	• TRC, or • dechlorination chemical (e.g. sulphites)	• Sample should be collected in free flowing areas. Stagnant areas should be avoided
Final Effluent	Grab	• *E. coli*	• Effluent *E. coli* densities should meet the effluent objectives specified in the C of A based on a monthly geometric mean • A grab sample should also be collected during a peak flow event to assess the efficiency of the system during high flows
Chlorination and Dechlorination Feed System	Flow	• Dosage	• Chemical dosages can be compared with values in Table 18.1 and Table 18.2 to assess the efficiency of disinfection

18.2.3 Common Problems and Potential Process Impacts

Table 18-5 presents the symptoms and causes of common problems encountered with the chlorination / dechlorination processes.

Table 18-5 - Chlorination / Dechlorination – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Poor secondary/tertiary effluent quality.	• Higher than typical chlorine dosages required • Poor effluent quality with respect to *E. coli* and/or TSS	• Upstream process upsets • Poor performance of secondary clarifiers (Chapter 14) • Poor performance of tertiary unit processes (Chapter 15)
Insufficient initial mixing of chlorination chemical.	• Higher than typical chlorine dosages required • Poor effluent quality with respect to *E. coli*	• Inadequate mixing energy available at chemical addition point (Section 18.2.5)
Insufficient initial mixing of dechlorination chemical.	• Higher than typical dechlorinating agent dosages • TRC residual detected after dechlorination	• Inadequate mixing energy available at chemical addition point (Section 18.2.5)
Insufficient contact time in contact chamber.	• Higher than typical chlorine dosages required • Poor effluent quality with respect to *E. coli*	• Short-circuiting or dead zones in contact chamber (Section 18.2.6) • Adequate plug flow conditions not achieved in contact chamber
Inadequate process control.	• Higher than typical chemical dosages required • Poor effluent quality with respect to *E. coli*	• Diurnal or seasonal variations in chlorine demand • Inaccurate TRC measurement

18.2.4 Optimizing Upstream Processes to Improve Disinfection Efficiency

Effluent quality has a strong impact on chlorine demand and disinfection efficiency. Generally, the characteristics of the effluent affect the efficiency of chlorine disinfection in two ways:

- exerting an additional chlorine demand thereby requiring a higher chlorine dosage to achieve the same level of pathogen reduction; and
- interference with the chlorination process.

Table 18-6 summarizes some of the impacts of effluent characteristics on the efficiency of chlorination / dechlorination processes.

Table 18-6 - Effects of Wastewater Quality on Chlorine Disinfection

Constituent	Effect on Chlorination Process
BOD_5, COD	• Organic compounds can exert a chlorine demand and reduce residual chlorine concentration
TSS	• Particles can shield bacteria from contact with chlorine • Particles can exert a chlorine demand and reduce residual chlorine concentration
Oils and Grease	• Can exert a chlorine demand and reduce residual chlorine concentration
Humic Materials	• May lead to the formation of chlorinated organic compounds that are measured as chlorine residuals but are not effective for disinfection
Ammonia	• Combines with chlorine to form chloramines that are less effective disinfectants. Therefore, a higher chlorine dosage is required compared to disinfection with free chlorine species
Nitrite	• Can be oxidized by chlorine increasing the chlorine demand • Can cause the formation of N-nitrosodimethylamine (NDMA), a disinfection by-product
Nitrate	• May lead to formation of NDMA
Iron / Manganese	• Can be oxidized by chlorine increasing the chlorine demand
pH	• Affects distribution between hypochlorous acid (HOCl) and hypochlorite ion (OCl^-) • The germidcical efficiency of HOCl is approximately 40 to 80 times greater than that of OCl^-. HOCl is the dominant species below pH of 7.5 and therefore, chlorination is more effective at lower pH values (WERF, 2008)

Table 18-6 – Effects of Wastewater Quality on Chlorine Disinfection (continued)

Constituent	Effect on Chlorination Process
Temperature	• Higher temperatures typically result in higher inactivation efficiencies
Alkalinity / Hardness	• No or minor effect
Industrial Discharges	• May lead to diurnal and seasonal variations in chlorine demand, depending on the effluent quality

Generally, optimizing upstream processes will optimize the efficiency of disinfection. That is, the better the quality of the effluent, the more efficient chlorination is for disinfection. Upstream process upsets that result in a deterioration of effluent quality will also reduce the efficiency of disinfection. In particular, high solids or soluble organic concentrations can increase the chlorine requirement to achieve the target *E. coli* densities.

In many cases, it may not be possible to change the characteristics of the sewage to improve the efficiency of disinfection. However, the following approaches can be used to optimize disinfection:

- Optimize biological treatment to improve effluent BOD_5 concentrations and/or achieve nitrification (Chapter 12);
- Optimize secondary clarification to improve effluent TSS concentrations (Chapter 14);
- Address causes of process upsets (e.g. rising sludge or filamentous bacteria) which may result in poor effluent quality and reduce the effectiveness of disinfection;
- Significant industrial flows to an STP typically increase the chlorine demand for disinfection. Variability in loading from industrial discharges can cause difficulty in predicting the required chlorine dosages. Incorporate measures such as monitoring industrial discharges and implementing source control programs, if required; and
- If the plant receives variable loading, consider implementing automated process control strategies to optimize the effectiveness of chlorination (Section 18.2.7).

18.2.5 Optimizing Initial Mixing to Improve Disinfection Efficiency

Chlorination

The disinfection effectiveness of chlorine is greatly enhanced by effective mixing of the effluent and chlorine solution. Proper mixing optimizes the disinfection process in the following ways:

- Optimizes the amount of contact between the chlorine and the pathogens in the water; and

- Avoids the formation of chlorine concentration gradients resulting in inefficient disinfection.

Initial mixing should take place in a fraction of a second. Efficient mixing by introducing chlorine in a highly turbulent regime can result in pathogen kills two orders of magnitude higher than when chlorine is added to a conventional rapid-mix reactor (Metcalf & Eddy, 2003).

Disinfection can be optimized by the installation of or improvements to chemical diffusers, mixing baffles or mechanical mixers, or other mechanisms to create a highly turbulent regime. In some instances, moving the chemical addition point to a more turbulent location can result in improved initial mixing.

If contact time is provided in the outfall pipe, providing mixing upstream of the outfall pipe will optimize contact of the effluent with the chlorinating agent, thereby optimizing the efficiency of the chlorination processes.

Tracer tests should be conducted to assess the degree of mixing available for both chlorination and dechlorination (Section 18.2.9).

Dechlorination

Inadequate mixing is more commonly observed in the dechlorination process than in the chlorination process. The dechlorination reaction is very rapid and requires contact with the full stream. Dechlorination chemicals are often added at the end of the contact chamber where there is little mixing energy. This can result in uneven distribution of dechlorination chemicals in the stream, and areas of flow that do not get adequately dechlorinated.

If mixing is inadequate, higher dosages of the dechlorinating chemical will be required, resulting in higher operating costs and poor performance.

Dechlorination can be optimized by the installation of diffusers and/or mechanical mixers, if inadequate mixing exists in the system.

18.2.6 Optimizing Contact Time to Improve Disinfection Efficiency

Chlorination

Sufficient contact time is required to optimize the inactivation of pathogens by maintaining contact between the target microorganisms and a minimum chlorine concentration for a specified period of time. According to the MOE Design Guidelines (2008), a minimum contact time of 30 minutes is required at design average daily flow, and 15 minutes at the design peak hourly flow.

Contact is provided in a chlorine contact chamber, which is typically designed as a serpentine chamber to create plug flow conditions. In some STPs, the outfall is used to provide some or all of the contact time.

Tracer tests should be conducted to verify that the required contact time is provided and ensure that there in no short-circuiting in the contact chamber (Section 18.2.9).

The following modifications can be incorporated to optimize contact time and prevent short-circuiting:

- Modify contact chambers to create plug flow conditions. Baffles or walls can be incorporated to create a serpentine flow configuration. Length-to-width ratios of at least 40:1 should be provided (MOE 2008; WERF, 2008);
- Provide rounded corners to reduce dead zone areas; and
- Ensure minimum velocities are maintained to prevent solids deposition in contact chamber (WEF/ASCE, 1998).

Chlorine contact chambers should also be cleaned regularly to ensure efficient performance.

As discussed in Section 18.2.5, if an outfall pipe is used to provide the necessary contact time, mixing should be provided upstream of the outfall pipe to optimize contact time of the effluent with the chlorinating agent.

Dechlorination

The dechlorination reaction occurs very rapidly. No additional tankage needs to be provided for dechlorination, if approximately 30 seconds of contact time is available in the effluent piping/channels at the design peak hourly flow. However, as discussed in Section 18.2.5, mixing at the addition point is often required.

18.2.7 Implementing Process Control Strategies to Optimize Disinfection

A chlorination / dechlorination system with manual control can be optimized by employing an automated feed control strategy to regulate the chlorination/ dechlorination chemical dosage. This approach will minimize chemical consumption and ensure effluent requirements are consistently met.

Systems for chlorination control typically consist of:

- Manual control: The operator adjusts dosages manually based on process conditions;

- Flow proportional (or open-loop) control: The chlorine feed rate is paced to the wastewater flow as measured by the plant's primary flow meter. Flow proportional control is sometimes referred to as feed-forward control;

- Automatic residual (or closed-loop) control: The chlorine dosage is controlled by the automatic measurement of the chlorine residual with an on-line chlorine analyzer. Residual control is sometimes referred to as feed-back control; and

- Automatic compound-loop control: The chlorine dosage is controlled by both the sewage flow and an automatic chlorine analyzer. The output from the wastewater flow meter and the residual analyzer is used by a programmable logic controller (PLC) to control chlorine dosage and residual.

The chlorine residual analyzer is a key piece of instrumentation available to optimize the chlorination disinfection process. Accurate measurement of TRC is important to ensure proper disinfection, while avoiding chemical waste and potential environmental impacts on receiving waters.

The analytical method adopted to monitor chlorine residual at an STP must be able to measure a range of concentrations with an appropriate level of accuracy and reproducibility.

Since on-line monitors are constantly submerged in the plant effluent, and the sample would contain particulate matter, potentially corrosive chemicals, and bacteria that will tend to grow on the equipment, an appropriate analyzer needs to be employed in order to achieve accurate measurements. In addition, proper maintenance and calibration, in accordance with manufacturer's instructions, must be conducted to ensure continued analyzer accuracy. Many on-line monitoring units may not be applicable to wastewater applications, or may only be useful for high quality tertiary effluents (WERF, 2008). The Instrumentation Testing Association (ITA) has tested chlorine analyzers for use in STPs. Test results are available through the ITA website www.instrument.org.

Similarly, typical control systems for dechlorination consist of:

- Flow proportional or feed forward control based on flow and TRC. The TRC is typically restricted to 0.02 mg/L in Ontario. Some devices can measure chlorine residuals down to levels as low as 1 µg/L; however, control at these levels may not be practical; and

- Slightly over dose the dechlorination chemicals. Measurement of a slight sulphite residual is an indication that the chlorine has been neutralized.

The dechlorinating agent should only be slightly overdosed to reduce costs and because some dechlorinating chemicals can adversely affect the receiving water (Environment Canada, 2003). The on-line dechlorination agent residual analyzer should be located at the point where the expected zero TRC would be monitored to limit the chance for bacterial growth and/or deposition on the analyzer.

18.2.8 Jar Testing

Jar testing should be conducted to estimate the optimal chlorine and dechlorinating agent chemical dosages for the available contact time. Jar testing should be conducted on the plant effluent over a range of chemical dosages and contact times to determine the optimal dosages.

Jar testing can also be conducted to verify that the plant is using the most cost-effective chemical at the plant, although other factors (e.g. ease of handling, ease of dosing, safety, availability, etc.) should be considered. There are several dechlorinating agents available as listed in Table 18.2, in addition to several emerging chemical disinfectants including peracetic acid, ferrate, and brominated compounds (WERF, 2008).

18.2.9 Tracer Testing

Tracer testing should be conducted to verify the hydraulic characteristics of the chlorine contact chamber. Tracer testing is conducted to determine the flow patterns through the contact chamber, and to identify any areas of short-circuiting, backmixing, or dead-space zones that would reduce the efficiency of disinfection.

Further information on tracer test methodology can be found in WEF/ASCE (1998).

18.2.10 Optimizing Chlorination / Dechlorination Disinfection

Table 18-7 summarizes potential symptoms, causes, and approaches to optimize effluent disinfection by chlorination/dechlorination.

Table 18-7 – Optimizing Chlorination / Dechlorination

Potential Symptom	Possible Problems and Causes	Possible Solutions
Elevated *E.coli* densities.	• Insufficient chlorine dosage	• Increase chlorine dosage • Verify that the sizes of chlorine pumps are adequate to deliver required chlorine dosages • If using using manual control, consider implementing automated chlorine control strategy
	• Industrial discharges leading to diurnal and seasonal variations in chlorine demand	• If using manual control, consider implementing automated chlorine control strategy
Higher than typical chlorine dosages required to achieve target *E. coli* densities.	• Excessive storage of sodium hypochlorite resulting in loss of strength	• Provide storage for a maximum of one month supply • Store sodium hypochlorite solution at appropriate temperature, and do not expose to sunlight
	• Poor effluent quality due to upstream processes contributing to higher TSS and/or parameters exerting an additional chlorine demand, such as ammonia in non-nitrifying or partially nitrifying STPs	• Optimize upstream processes • Poor performance of secondary clarifiers (Chapter 14) • Poor performance of tertiary unit processes (Chapter 15)
	• Inadequate contact time or short-circuiting in contact chamber	• Verify that there is 30 minutes of contact time at average daily flow and 15 minutes contact time during peak flow • Conduct tracer testing to check for short-circuiting and dead zones • Effective contact time can be achieved by employing baffles or serpentine flow configurations • To prevent short-circuiting, contact chambers should be plug flow, with length-to-width ratios of at least 40:1 (WERF, 2008)

Table 18-7 – Optimizing Chlorination / Dechlorination (continued)

Potential Symptom	Possible Problems and Causes	Possible Solutions
	• Insufficient mixing at point of chlorine addition	• Most mixing is done hydraulically through the contact chamber. Mechanical mixing should be implemented if there is not sufficient space for mixing and contact • Move chlorine addition point to a more turbulent zone • Provide chemical diffusers, mixing baffles, or mechanical mixers
	• Manual control of chlorine feed	• Implement process control strategies
	• Inaccurate TRC measurement / monitoring	• Ensure TRC analyzer system is clean • Ensure proper operation and maintenance practices are employed • Ensure type of meter is appropriate to measure parameter under the plant's conditions • Avoid using a strainer or filter in front of chlorine analyzer as it can lead to inaccurate TRC measurements and overdosing • Check location of meter. If used for chlorinator control, locate chlorine residual sample pumps to front end of first pass of contact chamber immediately after rapid mixing (Metcalf & Eddy, 2003), or 90-seconds travel time from injection point (WEF/ASCE, 1998)
Higher than typical dechlorinating dosages required.	• Insufficient mixing at point of dechlorination chemical addition	• Provide chemical diffusers, mixing baffles, or mechanical mixers
	• Limited reaction time available, requiring higher dechlorinating agent dosages to increase the rate of dechlorination reaction	• Increase dechlorination contact time, if possible, by modifying the location of the dechlorination agent injection point, or modifying weir levels. The injection point should be assessed for adequate mixing prior to any other adjustments

Table 18-7– Optimizing Chlorination / Dechlorination (continued)

Potential Symptom	Possible Problems and Causes	Possible Solutions
There is a chlorine residual after dechlorination.	• Insufficient dechlorination chemical added	• Provide larger feed pumps, or evaporators • Implement automated process control strategies
Formation and precipitation of light floc.	• Occurs most frequently in plants with alum addition. • Unreacted alum forms floc due to lowered pH in contact chamber that results from addition of chlorine	• Optimize alum addition (see Chapter 16)

18.3 ULTRAVIOLET (UV) IRRADIATION

18.3.1 Purpose and Mode of Disinfection

UV irradiation is becoming widely used in North America. UV light, in relatively low doses, is an effective STP effluent disinfectant. The primary mechanism of UV light inactivation is photochemical damage to the nucleic acids (DNA and RNA) in the microorganisms, rendering them unable to reproduce.

There are three main types of UV lamps available, which are classified by both their operating pressure and their output level: low pressure/low intensity (LP/LI), low pressure/high intensity (LP/HI), and medium pressure/high intensity (MP/HI) lamps.

Low pressure lamps are typically favoured for municipal STP disinfection applications in Ontario since they produce relatively monochromatic wavelengths in the germicidal range (WERF, 1995).

UV systems are typically designed based on the design peak flow, UV Transmittance (UVT), effluent TSS concentration, and effluent *E. coli* target density. UVT is the ability of the sewage effluent to transmit UV light. The UVT of treated effluent is affected by materials that can absorb or scatter UV radiation such as dissolved organic and inorganic compounds and suspended solids (WERF, 1995). The UVT of the effluent influences the UV demand, and affects the sizing of the system and possibly the configuration (spacing) of the lamps.

Table 18-8 presents typical UVT values and TSS concentrations for the different levels of effluent quality.

Table 18-8 - Typical UVT and TSS of Sewage Effluent

Effluent Type	UVT	TSS
Secondary Effluent	45 – 75 %	10 – 30 mg/L
Tertiary Effluent	60 – 80 %	5 – 10 mg/L

18.3.2 Evaluating Process Performance

The effectiveness of UV disinfection is measured by the ability to meet requirements for *E. coli* density in the plant effluent.

Table 18-9 summarizes the typical benchmarks for the UV disinfection.

Table 18-9 – Typical Benchmarks

Parameter	Benchmark
Effluent *E. coli* (based on monthly geometric mean density)	100 cfu / 100 mL (or as per C of A) (1)
Notes: 1. Typical effluent objectives seen in Cs of A issued in Ontario.	

Table 18-10 presents monitoring recommendations, in terms of sampling locations and analyses, in order to evaluate the performance of the UV disinfection process.

Sampling should be conducted daily over at least a one month period to obtain representative data. If on-line monitoring is available, data should be collected and trended over a sufficient period of time to assess the impact of diurnal and peak flows on the disinfection process.

Table 18-10 - UV Disinfection - Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
STP's Primary Flow Meter	On-line	Peak flow	Confirm peak flow through the UV system.
Secondary/Tertiary Effluent	On-line Grab	UVT TSS, *E. coli*	UVT should be measured continuously to assess any diurnal and seasonal variations related to

Table 18-10 - UV Disinfection - Recommended Process Monitoring to Evaluate Performance (continued)

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
			industrial discharges or other factors.
Final Effluent	Grab	*E. coli*	Effluent *E. coli* densities should meet the effluent objectives specified in the C of A based on a monthly geometric mean. A grab sample should also be collected during a peak flow event to assess the efficiency of the system during high flows.

18.3.3 Common Problems and Potential Process Impacts

Table 18-11 presents the symptoms and causes of common problems encountered with UV disinfection.

Table 18-11 – UV Disinfection – Symptoms and Causes of Common Problems

Problem	Common Symptoms and Potential Process Impacts	Common Causes
High effluent TSS concentrations.	• Poor effluent quality with respect to *E. coli* and/or TSS • Low UVT	• Upstream process upsets • Poor performance of secondary clarifiers (Chapter 14) • Poor performance of tertiary unit processes (Chapter 15)

Table 18-11 – UV Disinfection – Symptoms and Causes of Common Problems (continued)

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Low UVT of sewage.	• Reduced ability of water to transmit UV light • Poor effluent quality with respect to *E. coli*	• Due to wastewater characteristics, such as organic compounds (e.g. colouring agents, dyes, humic materials) and inorganic compounds (iron, manganese) • Iron has a high absorbancy of UV light. Iron is often used for chemical precipitation of phosphorus
Fouling and biofilms.	• Poor effluent quality with respect to *E. coli* • Reduces the intensity of the UV light that reaches the microorganisms • Higher UV dosage required	• Lamp fouling occurs due to the accumulation of inorganic, organic, and biological solids on the quartz sleeves that surround the lamp • Biofilms and algae growth can be a problem if the system is exposed to sunlight • Inadequate cleaning and maintenance
Poor system hydraulics.	• Poor effluent quality with respect to *E. coli.* • Reduces the average contact time resulting in ineffective disinfection	• Density currents causing flow to move along the bottom or top of the lamps • Entry and exit conditions that lead to the formation of eddy currents, thereby inducing uneven velocity profiles • Dead spaces or zones within the reactor reduce the effective reactor volume and shortens the average hydraulic retention time • System is hydraulically overloaded
Poor disinfection performance not attributable to problems identified above.	• Poor effluent quality with respect to *E. coli* and/or TSS	• Burned out bulbs • Operating at flows in excess of design peak flow capacity • Solid particle sizes in effluent large enough that bacteria are being shielded from the UV rays. Particle size distribution testing can be used to diagnose this problem

18.3.4 Optimizing Upstream Processes to Improve Disinfection Efficiency

Effluent quality can limit the effectiveness of UV disinfection. Generally, the characteristics of the effluent affect the efficiency of UV disinfection in three ways:

- Absorbing and/or scattering of UV light, thereby reducing the UV light that reaches the microorganisms;
- Shielding of microorganisms from exposure to UV light by suspended solids; and
- Contributing to fouling of quartz sleeves that surround the lamp, reducing the intensity of the UV light that reaches the microorganisms.

Table 18-12 summarizes some of the impacts of effluent characteristics on the efficiency of UV disinfection.

Table 18-12– Effects of Wastewater Quality on UV Disinfection

Constituent	Common Symptoms and Potential Process Impacts
BOD_5, COD	• No or minor effect unless specific organics absorb UV light
TSS	• Particles can scatter and absorb UV light reducing the UV light that reaches the microorganisms • Particles can shade microorganisms from UV light or bacteria can be embedded in large particles which may shield them from exposure to UV light • Can result in organic fouling of lamp sleeves
Oils and Grease	• Can result in organic fouling of lamp sleeves
Organic Compounds (e.g. colouring agents, dyes, and/or humic materials)	• Can absorb UV light, reducing UVT
Iron	• High absorbency of UV light in the germicidal range • Can adsorb onto suspended solids and bacterial clumps which can prevent UV light from penetrating and reaching embedded microorganisms • Can precipitate on quartz tubes
Manganese	• Can absorb UV light, reducing UVT

Table 18-12– Effects of Wastewater Quality on UV Disinfection (continued)

Constituent	Common Symptoms and Potential Process Impacts
pH	• Affects solubility of metals and carbonates that may absorb UV light
Total Dissolved Solids (TDS)	• Can cause scaling and the formation of mineral deposits on UV lamps
Alkalinity	• Can contribute to scaling • Affects solubility of metals that may absorb UV light
Hardness Metal Ions (such as iron, calcium, or magnesium)	• Can result in inorganic fouling of quartz sleeves that surround the lamp, reducing the intensity of the UV light that reaches the microorganisms
Industrial Discharges	• May lead to diurnal and seasonal variations in UVT, depending on the quality

The UVT of the sewage effluent may be variable. This may be attributed to industrial discharges that result in diurnal or seasonal variations in the UVT. Industrial discharges of inorganic or organic dyes, metals, and complex organic compounds may affect the UVT of the sewage. Stormwater inflows into the collection system can also reduce the UVT of sewage.

In many cases, it may not be possible to change the characteristics of the sewage to improve the efficiency of UV disinfection. However, the following approaches to optimize UV disinfection are available:

- Implement on-line monitoring of UVT to measure and document any diurnal or seasonal variations in UVT;

- Incorporate measures such as monitoring upstream industrial input and implementing source control programs, and addressing sources of infiltration, if required;

- Optimize secondary clarification to improve effluent TSS concentrations (Chapter 14);

- Address causes of process upsets (e.g. rising sludge or filamentous bacteria) which may result in poor effluent quality, reducing the effectiveness of UV disinfection; and

- The SRT of a system has some impact on the effectiveness of UV disinfection. As the SRT of the system is increased, the fraction of particles containing coliform bacteria is reduced (Metcalf & Eddy, 2003).

18.3.5 Minimizing Fouling and Biofilms to Optimize Disinfection

Fouling of quartz sleeves that surround the UV lamp will reduce the intensity of the UV light that reaches the microorganisms, thereby reducing the efficiency of UV disinfection. The total hardness, manganese and iron concentrations of the effluent are indicators of the potential for fouling of the UV lamps.

Lamp fouling can be caused by:

- the accumulation of inorganic, organic, and biological solids on the quartz sleeves that surround the lamp;
- high iron concentrations due to the addition of iron salts to the wastewater for the purposes of phosphorus removal;
- high levels of calcium and magnesium due to hard water;
- organic fouling can involve substances such as oil, grease, and suspended solids; and
- pH can affect the solubility of the scaling material.

To optimize performance, fouling can be controlled by mechanical, sonic or chemical cleaning units. Lamps should be regularly cleaned and maintained according to the manufacturer’s recommendations to maintain performance.

Exposure to light, even very dim light, can increase the occurrence of biofilms and fouling of exposed surfaces. As biofilms come off the surface, they can shield bacteria, reducing the effectiveness of UV disinfection.

Algae growth can be a problem if the system is exposed to sunlight. Clumps of algae can wrap around the lamps and decrease the amount of UV light that reaches the microorganisms.

To minimize the growth of biofilms and algae and optimize efficiency, UV channels should be completely covered. UV channels and equipment should be periodically cleaned using a suitable cleaning chemical, as recommended by the manufacturer.

18.3.6 Optimizing Reactor Hydraulics to Improve Disinfection Efficiency

The reactor hydraulics are a key factor in the performance of UV disinfection. Plug-flow conditions with radial mixing are required for efficient disinfection. Good radial mixing is required to prevent microorganisms from passing through the UV reactor between lamps and receiving a smaller UV dose than the average value. Radial turbulence is important because it ensures adequate mixing which minimizes the effects of short-circuiting and particle shading. These conditions are typically controlled by the reactor geometry, the lamp array geometry, and the flow rate of wastewater to the UV disinfection system. Due to the short contact time in UV systems, inlet and outlet conditions should be designed to optimize reactor hydraulics (WEF/ASCE, 1998).

Poor system hydraulics will reduce the efficiency of the UV disinfection process. Common hydraulic problems that result in short-circuiting include:

- density currents causing influent flow to move along the bottom or top of the lamps;
- entry and exit conditions that lead to the formation of eddy currents inducing uneven velocity profiles; and
- dead spaces or zones within the reactor, reducing the effective reactor volume and shortening the average hydraulic retention time.

Although it may not be possible to change the reactor hydraulics of an existing system, the following upgrades to optimize systems with poor hydraulics should be considered:

- provide submerged perforated diffusers;
- provide corner fillets in rectangular open-channel systems with horizontal lamp placement;
- provide flow deflectors in open-channel systems with vertical lamp placement; and/or
- provide serpentine effluent overflow weirs in combination with submerged perforated diffusers.

18.3.7 Collimated Beam Testing

Collimated beam tests can be conducted with effluent samples collected over a representative range of operating conditions to produce a dose-response relationship. The dose-response relationship established can be used to establish the required UV dose to meet the effluent requirements.

Plant operators or owners should contact their UV system supplier to discuss collimated beam testing.

18.3.8 Implementing Process Control Strategies to Optimize Disinfection

Process control strategies, such as flow pacing and dose pacing, can be used to optimize the performance of UV disinfection systems.

Flow pacing controls the lamp intensity and/or the number of banks of lamps in operation based on the flow rate through the UV disinfection system. This can reduce energy use during low flow periods. On-line flow monitoring equipment is required to implement flow pacing.

Dose pacing involves adjusting the lamp intensity and/or the number of lamps in operation based on not only the flow rate through the UV disinfection system, but also the UV intensity or UVT of the stream being treated. This ensures that a constant UV dose is being applied. Online UVT sensors and flow monitoring, or UV intensity sensors, are required to implement dose pacing.

Additional information regarding instrumentation and control strategies and requirements can be found in Hydromantis and Stantec (2003).

18.3.9 Optimizing UV Disinfection

Opportunities to optimize UV disinfection may be somewhat limited as several elements critical to the efficiency of UV disinfection are inherent in the equipment and the system design. As such, it is recommended that plant owners or operators contact the UV equipment manufacturer to discuss optimization opportunities. Table 18-13 summarizes potential problems and solutions to optimize UV disinfection.

Table 18-13 - Optimizing UV Disinfection

Possible Problems and Causes	Possible Solutions
Low UVT.	• Implement on-line monitoring of UVT to measure and document any diurnal or seasonal variations in UVT • Incorporate measures such as monitoring industrial discharges and implementing source control programs, and addressing sources of infiltration, if required
Poor effluent quality with respect to TSS.	• Optimize secondary clarification to improve effluent TSS concentrations (see Chapter 14) • Address causes of process upsets (e.g. rising sludge or filamentous bacteria) which may result in poor effluent quality and reduce the effectiveness of UV disinfection
Sleeve fouling.	• The total hardness, manganese and iron concentrations of the water are indicators of the potential for fouling of the UV lamp sleeves • Ensure units are regularly cleaned and maintained according to the manufacturer's recommendations • Avoid exposure of lamps to sunlight

Table 18-13 – Optimizing UV Disinfection (continued)

Possible Problems and Causes	Possible Solutions
Poor reactor hydraulics.	• Provide submerged perforated diffusers • Provide corner fillets in rectangular open-channel systems with horizontal lamp placement • Provide flow deflectors in open-channel systems with vertical lamp placement • Provide serpentine effluent overflow weirs in combination with submerged perforated diffusers
Poor process control.	• Implement flow pacing or dose pacing

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CHAPTER 19

SLUDGE TREATMENT PROCESSES

CHAPTER 19

SLUDGE TREATMENT PROCESSES

19.1 AEROBIC DIGESTION

19.1.1 Purpose and Types of Aerobic Digesters

Aerobic digestion is a biological suspended growth sludge stabilization process in which the microorganisms operate in an endogenous state and consume their own cell tissue. The objective of aerobic digestion is to generate stabilized, digested sludge (biosolids) resulting in a reduction in sludge mass, pathogens content, odour production potential and vector attraction potential. Typically, aerobic digestion is utilized to stabilize WAS from extended aeration sewage treatment plants which do not have primary clarification.

Aerobic digesters can operate in batch or continuous modes. Aerobic digestion normally takes place in open tanks; however, in cold climates, sheltering or covering of tanks can limit heat loss, lower the retention time required and/or improve the efficiency of the process. The majority of aerobic digestion processes in Ontario operate at mesophilic temperatures (~10 to 30 ^{o}C).

Autothermal thermophilic aerobic digestion (ATAD) is a process in which digestion takes place at thermophilic temperatures (55 ^{o}C). The higher temperature allows for increased rates of volatile solids and pathogens destruction at shorter hydraulic retention times. The process is termed autothermal as the sludge is pre-thickened and fed to the digester at higher loading concentrations. At these higher concentrations, excess energy is produced as a result of the exothermic biological oxidation reactions, resulting in heat generation for the process. Over the past several years, there have been ATAD processes implemented in Ontario at sewage treatment plants including Long Sault, Cardinal and Morrisburg.

Further information on the design and operation of aerobic digesters can be found in MOE (2008), WEF (1995) and Metcalf & Eddy (2003).

19.1.2 Evaluating Process Performance

Table 19-1 presents recommended monitoring, in terms of sampling locations and analyses, in order to evaluate the performance of the aerobic digestion process.

Table 19-1 – Aerobic Digestion – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Influent Sludge	Composite Recommended	• Flow rate • COD • TS or TSS • VS or VSS	In the absence of inlet sampling, WAS characteristics can be substituted if this is the sole or major feed source to the digester.
Within Digester	Grab	• Temperature • pH • DO	Several readings should be taken at a number of representative locations within the digester.
Supernatant	Composite Recommended	• Flowrate • BOD_5 • TSS • VSS • TAN • NO_x-N • TP	
Digested Sludge (Biosolids)	Grab	• Volume • COD • TS or TSS • VS or VSS • SOUR	Metals, pathogens, nutrients and other regulated parameters should also be monitored if land application is used for biosolids management.

In addition to the recommended sample locations and analyses presented in Table 19-1, it is recommended that the following also be monitored:

- Characteristics of sludge streams entering digesters (if more than one);
- Volatile solids loading rate (kg VS/m^3 of digester volume per day); and
- SRT of each digester.

Figure 19-1 presents a process schematic of a typical aerobic digestion process, along with recommended sampling locations.

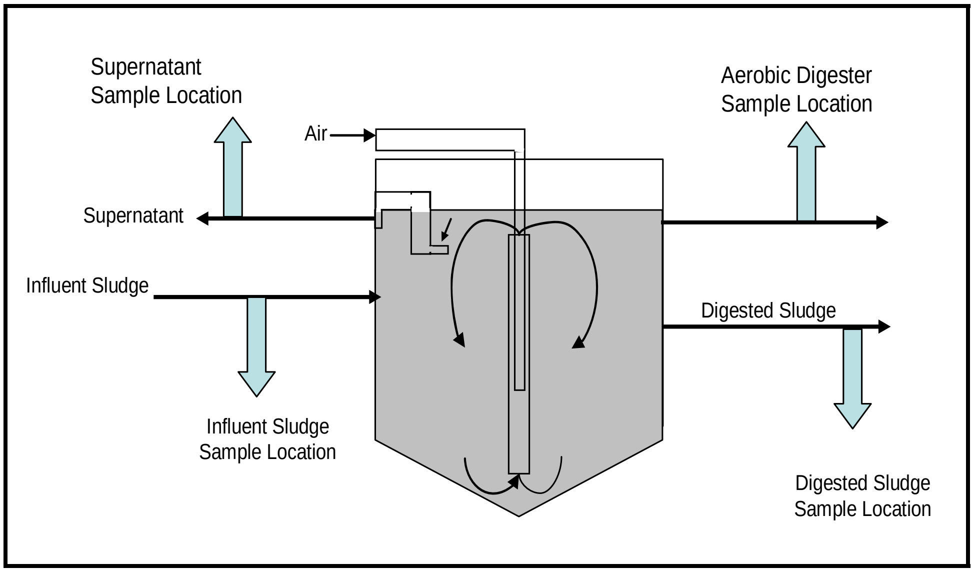

Figure 19-1 – Aerobic Digestion – Process Schematic and Recommended Sampling Locations

Typically, aerobic digester performance is evaluated based on the achieved VS or VSS destruction and the effectiveness of pathogens inactivation.

Table 19-2 presents typical process performance for the aerobic digestion process.

Table 19-2 - Aerobic Digestion - Typical Process Loading and Performance

(Adapted from MOE, 2008)

Operating Condition	VS Loading ($kg/m^3 \cdot d$)	VSS Reduction (%)	DO Concentration (mg/L)	SRT (d)	Operating Temperature (°C)
Mesophilic Aerobic Digestion	1.6[1]	38-50[2]	1-2	45	35-38

Notes:

1. To the first digester.
2. Metcalf & Eddy (2003).

19.1.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the aerobic digestion process are shown in Table 19-3.

Table 19-3 – Aerobic Digestion – Common Problems and Impacts

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Inert solids accumulation.	• Lower than expected VS reduction	• Poor grit removal (Chapter 10) • Feeding of primary sludge to digester (Section 19.1.4)
Poor VS destruction.	• High outlet VS concentration • Lower than expected VS reduction	• Inadequate mixing (Section 19.1.5) • Low operating temperature • Low SRT • Low DO (Section 19.1.6)
Digester foaming.	• Visible foam • High TSS in the supernatant	• Filamentous organisms in secondary treatment • Hydraulic or volatile solids overloading • Seasonal temperature change
Poor thickening or dewatering characteristics of aerobically digested sludge.	• Lower TS concentration than expected from dewatering process	• Excessive aerobic digestion mixing intensity (Section 19.1.5) • Excessive aerobic digestion SRT

19.1.4 Options to Enhance Stabilization

Enhancing the aerobic digestion process involves ensuring that the operating conditions of the process are optimal in terms of temperature, pH, mixing intensity, and DO concentration as discussed in MOE (2008). In order to confirm the actual operating conditions, an intensive sampling program may be required to assess the process conditions and check the uniformity of mixing, temperature and oxygen concentration throughout the process (Sections 19.1.5 and 19.1.6).

As aerobic digestion is similar to the extended aeration process, the methods used to optimize that process can also be applied to enhance the aerobic digestion process. Further information on enhancing the performance of biological suspended activated sludge processes is provided in Chapter 12.

Thickening of solids before aerobic digestion results in longer retention times, lower digester volume requirements, less decanting requirements, and ultimately higher volatile solids destruction. Further information on sludge thickening can be found in Section 19.4. Care should be taken to ensure that the digester feed solids concentration is not increased to a level where autothermal heating can occur unless measures are taken to manage foam production and odour generation that are characteristic of ATAD processes.

Minimizing or eliminating the primary sludge input to the digesters can improve stabilization. Primary sludge contains a higher concentration of inert solids that take up room within the digester and are not destroyed. Removing these from the feed stream can increase the digester capacity and improve stabilization. In addition, the presence of primary sludge in an aerobic digester feed stream will significantly increase oxygen demand and energy use in the process.

In the case of high inert solids content within the WAS stream, optimization of the preliminary treatment processes should be undertaken to decrease the amount of grit that is present in the sludge (Chapter 10).

Temperature is one of the key operating parameters for aerobic digestion as biological activity decreases significantly at lower temperatures. Minimizing heat loss in the aerobic digestion process through insulation and/or partial covering will improve biosolids stabilization.

19.1.5 Mixing Testing

Mixing within aerobic digesters is typically provided by diffused air aeration systems. The aeration requirements are based on providing adequate oxygen transfer to maintain the DO level between 1 and 2 mg/L and ensuring complete mixing of the solids in suspension. Further information on mixing requirements can be found in MOE (2008).

In order to determine the efficiency of digester mixing, tracer tests can be performed on the digester using techniques similar to those discussed for biological treatment processes in Section 12.9.1.

If the results of the testing indicate that the mixing is not optimal, further investigation should be undertaken to determine the cause. Field measurements to define TS and DO concentration profiles in the digester can be performed to determine if there is evidence of regions within the digester that have insufficient mixing. Maintenance may be required on the aeration system. Clogged aerators or poorly performing air distribution grids can negatively impact the mixing patterns throughout the digester. Refer to Chapter 13 for details on optimizing aeration systems.

In some cases, poor air distribution can be improved by having dedicated blowers for the digester separate from the blowers that provide air to the extended aeration bioreactors. Where the air supply to the digester is provided from the same header as the air supply to the sewage treatment bioreactors, changes in head due to water level in the digesters when decanting or differences in the diffuser characteristics can impact the air flow distribution in the system, affecting both mixing (Section 19.1.5) and oxygen transfer (Section 19.1.6).

In some cases, replacement of the aeration system or installation of supplemental mechanical aeration may be required if the aeration system is not able to provide adequate mixing.

19.1.6 Oxygen Transfer Testing

The methods employed to test for oxygen transfer within aerobic digesters are the same as those within biological processes for sewage treatment. Information on measuring oxygen transfer can be found in Section 13.4.

If the results of oxygen transfer testing indicate that there is inadequate oxygen transfer to the aerobic digestion process, optimization of the aeration system may be required. Information on optimizing aeration systems can be found in Chapter 13.

19.2 ANAEROBIC DIGESTION

19.2.1 Purpose and Types of Anaerobic Digesters

Anaerobic digestion is a commonly utilized method to stabilize sludge, reduce pathogens, reduce biomass quantity by partial destruction of volatile solids (VS), and produce a useable gas (primarily methane) as a by-product. Different naturally occurring microbial populations are responsible for the three stages of anaerobic digestion: hydrolysis, volatile acid fermentation and methane formation. The microorganisms required for hydrolysis and volatile acid formation are fairly robust in comparison to the methanogens required for methane formation (WEF, 1995). For this reason, anaerobic digesters are typically operated under environmental conditions that favour the growth of methanogens. Methanogens can be sensitive to pH, temperature and sludge composition.

Anaerobic digestion processes are categorized based on the operating temperature: mesophilic (35 ^{o}C) or thermophilic (55 ^{o}C). Anaerobic digesters are also classified based on mixing intensity. High rate digesters are those which have mixing (gas or mechanical) while low rate digesters are unmixed.

Two-stage mesophilic anaerobic digestion is the most common digestion process for large sewage treatment plants in Ontario (MOE, 2008). The process layout is known as high rate digestion and includes a heated and mixed primary digester followed by an unheated and unmixed secondary digester. For larger plants, more than one digester can be required in each stage.

The thermophilic digestion process resembles mesophilic anaerobic digestion with the exception of a higher operating temperature (55 ^{o}C). The higher operating temperature can result in higher pathogen destruction and shorter required retention times for volatile solids reduction. Other anaerobic digestion processes include temperature phased anaerobic digestion (TPAD) in which the first stage is operated as a thermophilic digester and the second stage is operated as a mesophilic digester.

Further information on the design and operation of anaerobic digestion processes can be found in MOE (2008), WEF/ASCE (1998), Metcalf & Eddy (2003) and WEF (1995).

19.2.2 Evaluating Process Performance

Table 19-4 presents recommended monitoring, in terms of sampling locations and analyses, in order to evaluate the performance of the anaerobic digestion process.

Table 19-4 – Anaerobic Digesters – Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Influent Sludge	Composite Recommended	• Flow rate • COD • TS or TSS • VS or VSS	In the absence of inlet sampling, the characteristics of the WAS and raw sludge streams (or co-thickened sludge stream) can be used if these are the sole or major feed sources to the digester.
Within Digester	Grab	• Temperature • pH • Alkalinity • VFAs	Several readings should be taken at a number of representative locations within the primary digester.
Supernatant	Composite Recommended	• Flowrate • BOD_5 • TS or TSS • VS or VSS • TKN • TAN • TP	
Digested Sludge (Biosolids)	Grab	• Volume • COD • TS or TSS • VS or VSS	Metals, pathogens, nutrients and other regulated parameters should also be monitored if land application is used for biosolids management.

In addition to the recommended sample locations and analyses presented in Table 19-4, it is recommended that the following also be monitored:

- Volatile solids loading rate (kg VS/m^3 of digester volume per day);
- SRT within the digester(s);
- HRT within the digester(s); and
- Digester gas flow and composition (CH_4 and CO_2 content).

Figure 19-2 presents a process schematic of a high rate anaerobic digestion process, along with recommended sampling locations.

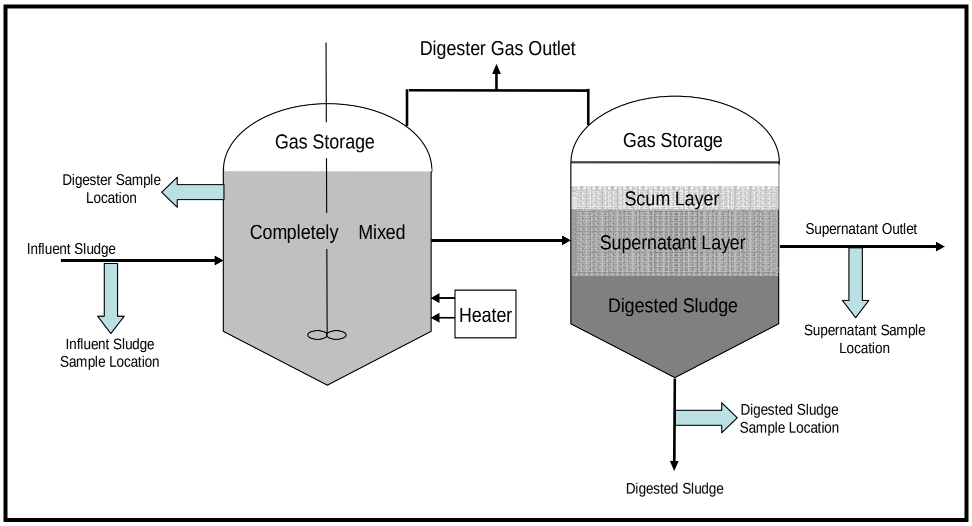

Figure 19-2 - Anaerobic Sludge Digestion - Process Schematic and Recommended Sampling Locations

Typically, anaerobic digester performance is evaluated based on the achieved VS or VSS destruction, the effectiveness of pathogen inactivation and the quantity and quality of gas produced.

Table 19-5 presents typical process performance for the mesophilic anaerobic digestion process.

Table 19-5 - Mesophilic Anaerobic Digestion - Typical Process Loading and Performance

(Adapted from MOE, 2008)

Operating Condition	VS Loading ($kg/m^3 \cdot d$)	VSS Reduction (%)	Minimum HRT (d)	Minimum SRT (d)	Operating Temperature (^oC)
Mesophilic Anaerobic Digestion	1.6[(1)]	56-65[(2)]	≥15	≥15	35

Notes:

1. To high rate primary digester.
2. Metcalf & Eddy (2003).

19.2.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the anaerobic digestion process are shown in Table 19-6.

Table 19-6 - Anaerobic Digestion Processes – Common Problems and Impacts

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Inert solids accumulation.	• Lower than expected VS reduction	• Poor grit removal (Chapter 10)
Poor VS destruction.	• Lower than expected VS reduction • Lower than expected digester gas production	• Presence of inhibitory compounds (Section 19.2.4) • Low alkalinity (Section 19.2.4) • pH not in optimum range (Section 19.2.4) • Inadequate mixing (Section 19.2.5) • Low or fluctuating operating temperature (Section 19.2.4) • Inconsistent hydraulic and/or solids loading
Digester foaming.	• Visible foam • High TSS in the supernatant	• Filamentous organisms in secondary treatment (Section 12.2) • Hydraulic or solids overloading (Section 19.2.4)

19.2.4 Options to Enhance Stabilization

Enhancing the anaerobic digestion process involves ensuring that the operating conditions of the process are optimal in terms of temperature, pH, mixing intensity, and alkalinity concentration as discussed in MOE (2008). In order to confirm the actual operating conditions, an intensive sampling program may be required to assess the process conditions and check the uniformity of mixing, and temperature throughout the digesters.

Stabilization can be limited by the presence of inhibitory substances. Inhibitory substances can include heavy metals, sulphides, and concentrations of free ammonia over 80 mg/L (WERF, 2009). As methanogens are more sensitive to inhibition than the organisms responsible for hydrolysis and acid formation, an increase in VFA concentrations and a reduction in gas production or methane content of the biogas may be indicative of an inhibitory effect.

Thickening of the sludge prior to digestion can optimize anaerobic digestion by increasing the solids retention time within the digester. Further information on thickening of solids can be found in Section 19.4.

The presence of a large amount of inert solids can waste valuable digester volume by accumulating within the digester. Optimization of the preliminary treatment stage may be required to minimize the presence of inert solids in the digester. Information on optimizing preliminary treatment can be found in Chapter 10. In the event of excessive accumulation of inert solids, physical removal of the solids through a digester clean-out may be required.

Minimizing any temperature fluctuations can improve digestion as rapid changes of even of a degree or two can inhibit anaerobic digestion. The digester should be fed at regular and frequent intervals to minimize temperature fluctuations. If there are multiple primary digesters, the feed should be equally distributed among the digester tanks to equalize loading and minimize temperature fluctuations.

Where possible, increasing the operating temperature of the digester will increase the reaction rate and improve pathogens and VS destruction and decrease the SRT required for the same level of stabilization.

19.2.5 Mixing Testing

Mixing within anaerobic digesters is typically provided mechanically by mixers or recirculating pumps or by biogas recirculation.

To determine the efficiency of the mixing and the presence of dead space in the digestion tank due to accumulated grit or other inert material, digester tracer tests should be performed. Tracer testing is discussed in Section 12.9.1. In conducting digester mixing tests, care should be taken to ensure that an appropriate tracer is used that will not be adsorbed or degraded by the digester and can be readily measured in the concentrated sludge streams. Lithium chloride is a commonly used tracer for digester mixing tests.

19.2.6 Temperature and Solids Profiling

Temperature profiling entails temperature monitoring at various locations throughout the digester. The purpose of temperature profiling is to confirm the actual operating condition as well as determine any regions in the digester that have a temperature gradient.

Solids profiling involves developing a mass balance of all the solids entering and leaving the digester. An inventory of the solids within the feed stream(s) and leaving in the digested sludge as well as in the supernatant that is recycled to the STP liquid train should be included. Solids profiling provides information needed to assess the efficiency of the digestion process as well as in determining the sludge age and organic loading to the process (WERF, 2009).

Solids profiling within the digester in a manner similar to temperature profiling can also be used to assess the effectiveness of mixing in the digester, and identify areas where grit and other heavy inert material have accumulated leading to reduced effective digestion volume and retention time. Solids profiling within the digester will provide an indication of the need to clean out a digester to recover lost digestion volume.

19.2.7 Gas Production Monitoring

Monitoring of the gas produced by anaerobic digestion is commonly used to measure the effectiveness of the digestion process. The gas by-product can be used as a fuel source to heat the digester or to produce energy.

The main components of digester gas (biogas) are methane (CH_4) and carbon dioxide (CO_2) with trace amounts of nitrogen and hydrogen sulphide. An optimally operating anaerobic digester typically produces 65 percent methane and 35 percent carbon dioxide along with trace amounts of other gases, with a heating value of 5,850 kg-cal/m^3 (656 Btu/ft^3) (Wang *et al.*, 2007; MOE, 2002). During periods of digester upset, an increase of carbon dioxide and a reduction in methane content is normally evident.

Information on gas monitoring can be found in WERF (2009), Metcalf & Eddy (2003), Wang *et al.* (2007) and MOE (2002).

19.3 OTHER SLUDGE TREATMENT PROCESSES

Although digestion processes (aerobic or anaerobic) are the most commonly used sludge treatment processes in Ontario, several other treatment processes are available including:

- Alkaline stabilization;
- Thermal drying;
- Composting;
- Pelletization; and
- Incineration.

Most of these processes are proprietary. The technology manufacturer/supplier should be contacted regarding optimization of these processes.

19.4 SLUDGE THICKENING

19.4.1 Purpose and Types of Sludge Thickeners

Sludge thickening is the process of removing free water not bound within the sludge flocs. The result of removing a portion of the free water is a higher solids content, typically between 4 and 14 percent depending on the type of thickener. Thickening is typically undertaken in order to reduce the downstream digester volume and heating required to reach the same solids retention time.

There are a number of types of sludge thickeners that utilize different mechanisms to increase the solids concentrations of the sludge including; gravity settlers, gravity belt thickeners (GBTs), rotary drum thickeners (RDTs), thickening centrifuges, and dissolved air flotation (DAF) thickeners.

Gravity settlers use settling processes usually accompanied by a slowly revolving sludge collector. GBTs thicken sludge by placing the sludge in between two fabric belts which move and allow the water to separate from the sludge by gravity. RDTs act by straining free water from the sludge through a rotating cylindrical screen.

Thickening centrifuges apply a strong centrifugal force to the sludge which separates the sludge and water as a result of the density differences. The lighter liquids remain near the center of rotation and exit by overflowing a weir. There are three types of centrifuges; basket, solid-bowl and disc centrifuges. Basket centrifuges are rotating vertical chambers with a weir at the top. Solid-bowl centrifuges bring sludge into a fast rotating bowl using a screw-type conveyor. Within the bowl the solids move to the walls while the liquid is decanted or drawn-off prior to removing the solids from the wall. Disc centrifuges involve feeding the sludge in the centrifuge, either at the top or bottom, where a rotor distributes the sludge to an outer chamber. The solids move toward the wall where stacks of discs are located that collect the liquid. The collected liquid then flows to a discharge chamber. Solid bowl centrifuges are most commonly used for sludge thickening.

Thickening of sludge using DAF occurs by introducing air to the sludge in a unit that has an elevated pressure. When the sludge is depressurized, fine air bubbles are formed which carry sludge to the surface where it can be removed.

Further information on the purpose and type of thickeners can be found in MOE (2008), Wang *et al.* (2007) and Metcalf & Eddy (2003).

19.4.2 Evaluating Process Performance

Table 19-7 presents recommended monitoring, in terms of sampling locations and analyses, in order to evaluate the performance of sludge thickeners.

Table 19-7 - Sludge Thickening - Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses	Comments
Influent Sludge	Composite Recommended	• Flowrate • SVI • TS	In the absence of inlet sampling, WAS characteristics can be substituted if this is the sole or major feed source to the thickener.
Centrate/Supernatant/ Subnatant	Composite Recommended	• Flowrate • TS • TSS • TKN • TAN • BOD_5 • TP	
Thickened Sludge	Composite Recommended	• Flowrate • TS	

In addition to the recommended sample locations and analyses presented in Table 19-7, it is recommended that the overflow and underflow rates within the thickeners (if applicable) be monitored.

Figure 19-3 presents a process schematic of a sludge thickening process, along with recommended sampling locations.

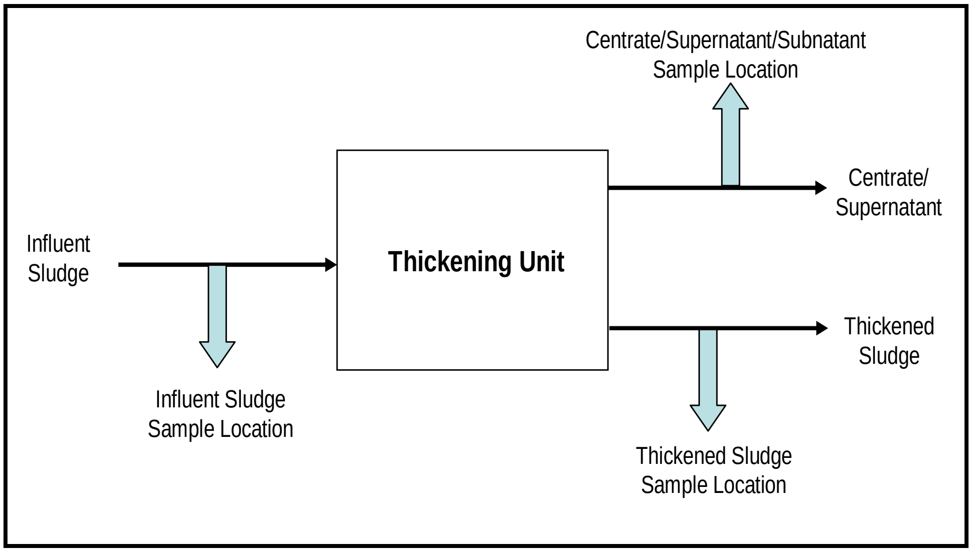

Figure 19-3 - Sludge Thickening - Process Schematic and Recommended Sampling Locations

Typically, sludge thickener performance is evaluated based on the solids captured and the total solids content achieved.

Table 19-8 presents typical process performance for the various types of sludge thickeners along with the sludge type usually thickened using that method.

Table 19-8 - Sludge Thickening - Typical Process Performance

(Adapted from MOE, 2008 and Metcalf & Eddy, 2003)

Thickening Process	Sludge Type	Expected Performance	
		Total Solids (%)	Solids Capture (%)
Basket Centrifuges	WAS with polymer (1)	8-10	80-90
Disc-nozzle Centrifuges	WAS with polymer (1)	4-6	80-90
Solid Bowl Centrifuges	WAS with polymer (1)	5-8	70-90
GBT	WAS with polymer	4-8	≥ 95
RDT	Raw Primary	7-9	93-98
	WAS with polymer	4-8	≥ 95
	Raw primary and WAS	5-9	93-98
Gravity Thickeners	Raw primary	8-10	n/a
	Raw primary and WAS	4-8	n/a
	WAS	2-3 (2)	n/a
	Digested primary	5-10	n/a
DAF	WAS	4-6	≥ 95 (3)

Notes:

n/a – not applicable

1. Reduced solids concentration expected without use of polymers.
2. Improved results reported for oxygen rich activated sludge.
3. Using flotation aids.

19.4.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with the sludge thickening process are shown in Table 19-9.

Table 19-9 – Sludge Thickening – Common Problems and Impacts

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Thickened sludge has low solids content.	• Lower than expected TS in the thickened sludge • Higher than expected TS in the centrate/supernatant/subnatant	• Inadequate polymer dosing (Section 19.4.4) • Inadequate upstream sludge storage • Thickener is hydraulically overloaded due to poor feed pump controls • Short-circuiting through the thickener • Feeding primary and secondary sludge separately to the thickener
Septic thickened sludge.	• Thickened sludge is odorous • High sludge blanket (gravity thickeners) • Floating of sludge (gravity thickeners)	• Ineffective pump controls resulting inconsistent or infrequent sludge feeding • Low hydraulic overflow or underflow rate • Long retention time of solids within thickener

19.4.4 Options to Enhance Thickening

Optimizing the performance of thickeners can involve ensuring that the operation of the unit is as close to the manufacturer's recommended operating conditions as possible. Consultation with the process supplier can be useful in ensuring that the unit is operating optimally.

In addition, thickening can be improved by optimizing the use of polymers. Dosing polymers into the process along with the feed or dosing at various points can improve the solids capture and increase the solids content in the thickened sludge. Both the polymer dosage and dosing point(s) should be reviewed as in some cases multiple dosing points can improve performance. Full scale tests should be performed in order to optimize polymer dosage. As polymer effectiveness depends on the polymer dose per mass of solids (mg of polymer per kg dry solids in the sludge feed) not on dose per litre of sludge flow, dosing polymer based on flow only will not be optimal unless sludge concentration is relatively constant. In order to optimize polymer type and mixing rate, jar testing should be performed.

Thickening can be improved by ensuring that influent flows and concentrations are maintained relatively constant which will prevent wide variations in solids load and polymer dose. Minimizing the variability of feed flows and concentrations can be accomplished a number of ways including the implementation of online instrumentation and control systems that can measure feed solids concentration and flow. In addition, implementation of mixed storage tanks prior to mechanical thickening equipment can also be used to minimize variability rather than feeding directly from clarifier underflows to thickeners.

Stress testing of the thickening process can be undertaken in order to determine the maximum throughput, optimal operating settings, polymer dosage requirements, and the impact on the thickened sludge concentration and centrate/supernatant/subnatant quality. An example of thickening process stress testing is presented in Section 19.7.1.

Further information on enhancing thickening processes can be found in WERF (2009).

19.5 SLUDGE DEWATERING

19.5.1 Purpose and Types of Sludge Dewatering

The purpose of dewatering is to remove the floc-bound and capillary water from sludge and biosolids prior to further processing or off-site disposal. Sludge dewatering is similar to sludge thickening with the main difference in the solids content of the end product which is much higher in dewatered sludge/biosolids. In order to improve sludge dewatering, chemical conditioning is typically used to improve the solids capture and increase the solids content in the dewatered sludge.

There are numerous dewatering processes available, some of which are similar to those processes used for thickening. A number of types of sludge dewatering processes can be employed to increase the solids content of the sludge to between 10 to 50 percent depending on the process. The processes include: solid bowl centrifuges, belt filters presses, filter presses, and vacuum filters. Solids bowl centrifuges are described in Section 19.4.1.

Belt filter presses are continuously fed units that dewater chemically conditioned sludge first in a gravity drainage section where the free water is removed. After the free water is removed, low pressure is applied by porous belts to remove a portion of the bound water from the sludge. Filter presses dewater by the application of high pressure to remove bound water. Vacuum filters remove water from sludge by application of a vacuum.

Further information on the purpose and type of dewatering processes can be found in MOE (2008), Wang *et al.* (2007) and Metcalf & Eddy (2003).

19.5.2 Evaluating Process Performance

Table 19-10 presents recommended monitoring, in terms of sampling locations and analyses, in order to evaluate the performance of dewatering processes.

Table 19-10 - Sludge Dewatering - Recommended Process Monitoring to Evaluate Performance

Location	Types of Sample / Measurement	Parameters / Analyses
Influent Sludge	Composite Recommended	• Flowrate • TS
Centrate/Filtrate	Composite Recommended	• Flowrate • TS • TSS • TKN • TAN • BOD_5 • TP
Dewatered Sludge	Composite Recommended	• Flowrate • TS • VS

Figure 19-4 presents a process schematic of a dewatering unit, along with recommended sampling locations.

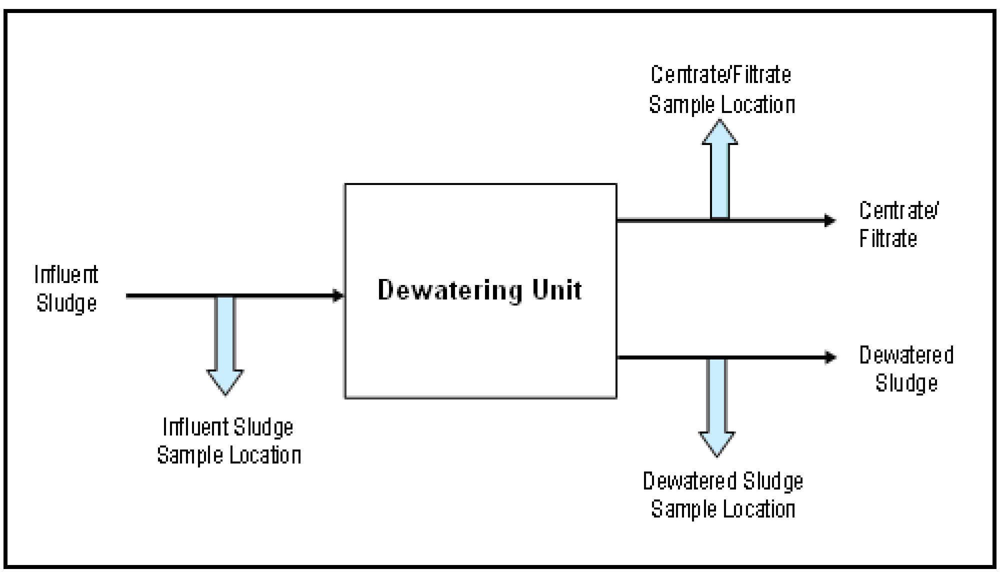

Figure 19-4 - Sludge Dewatering - Process Schematic and Recommended Sampling Locations

Typically, sludge dewatering process performance is evaluated based on the solids captured and the total solids content achieved.

Table 19-11 presents typical process performance for the various types of sludge dewatering processes along with the sludge type usually dewatered using that method.

Table 19-11 - Sludge Dewatering - Typical Process Performance

(Adapted from MOE, 2008 and Fournier Inc., 2010)

Dewatering Process	Sludge Type	Expected Performance	
		Total Solids (%)(1)	Solids Capture (%)
Solid Bowl Centrifuges	Undigested primary plus WAS	15-30	95-99
	Digested primary plus WAS	15-30	95-99
	WAS	12-15	95-99
Belt Filter Press	Undigested primary plus WAS	14-25	85-95
	Digested primary plus WAS	14-25	85-95
	WAS	10-15	85-95
Filter Press	Undigested primary plus WAS	30-50	90-95
	Digested primary plus WAS	35-50	90-95
	WAS	25-50	90-95
Vacuum Filter	Undigested primary plus WAS	10-25	90-95
	Digested primary plus WAS	15-20	90-95
	WAS	8-12	90-95
Rotary Press	-	-	Up to 95%

Notes:

1. Values presented in this table assume the use of conditioning chemicals (i.e. polymers). If no conditioning chemicals are used, cake solids and solids capture values may be reduced.

19.5.3 Common Problems and Potential Process Impacts

Symptoms and causes of common problems encountered with sludge dewatering processes are shown in Table 19-12.

Table 19-12 - Sludge Dewatering - Common Problems and Impacts

Problem	Common Symptoms and Potential Process Impacts	Common Causes
Dewatered sludge has low solids content	• Lower than expected TS in the dewatered sludge • Higher than expected TS in the centrate/filtrate	• Inadequate polymer dosing (Section 19.5.4) • Dewatering process is hydraulically overloaded due to poor feed control (Section 8.2) • Short-circuiting through the unit
Septic dewatered sludge	• Dewatered sludge is odorous	• Ineffective pump controls resulting inconsistent or infrequent sludge feeding (Section 8.2) • Low hydraulic overflow or underflow rate • Long retention time of solids within unit

19.5.4 Options to Enhance Dewatering

As dewatering is a process similar to thickening, optimizing dewatering process performance involves similar techniques. Possible techniques to enhance dewatering are listed below with additional information available in Section 19.4.4:

- Consultation with the process equipment manufacturer/supplier to ensure that there are no equipment or operating issues;
- Jar testing to optimize polymer type and mixing rate;
- Full scale studies to optimize polymer dosing locations and dosage;
- Installation of online instrumentation to measure and control the feed flow and solids density to minimize feed fluctuations;
- Implement mixed storage tanks prior to dewatering units to minimize feed variability; and

- Stress testing to determine optimal operating settings and maximum throughput capacity.

Further information on enhancing dewatering processes can be found in WERF (2009).

19.6 SLUDGE STORAGE

19.6.1 Purpose and Types of Storage

Storage of sludge or biosolids may be needed prior to processing, utilization, or disposal. The type of storage facility utilized depends on the solids content and/or level of stabilization. There are numerous types of sludge storage facilities which can include a combination of:

- Sludge drying beds;
- Lagoons;
- Separate tanks;
- Pad area for dewatered and dried sludge; and
- Additional capacity available within stabilization units.

In Ontario, storage for a minimum of 240 days worth of biosolids generation is considered to be best practice when associated with a land application program.

19.6.2 Maximizing Available Storage Capacity

Optimizing storage capacity involves maximizing the solids content of the sludge by optimization of thickening and dewatering processes (Sections 19.4 and 19.5).

In some cases, additional sludge storage can be realized by utilizing idle tanks or digesters that are not required to treat current sewage flows.

19.7 CASE HISTORIES

19.7.1 Solids Thickening and Dewatering Optimization at the Robert O. Pickard Environmental Centre (ROPEC) in Ottawa

The following case study is based on information presented in Newbigging *et al.* (2006).

The Ottawa ROPEC is a secondary treatment facility with a design average flow of 545,000 m^3/d and design peak flow of 1,362,000 m^3/d. Currently, the plant is operating at approximately 80 percent of capacity (439,000 m^3/d) with effluent quality able to meet all effluent requirements. The plant consists of raw sewage pumping, pretreatment, primary clarification, suspended growth activated sludge, final clarification and disinfection. Solids handling includes thickening and dewatering centrifuges along with anaerobic digestion and bio-gas utilization through co-generation.

Experience at the plant suggested that there might be additional capacity available in the thickening and dewatering centrifugation processes. An optimization study was undertaken to establish the actual capacity of each unit process. Evaluation, modifications and stress testing of the centrifuges were conducted to determine the ultimate capacity, as well as identify any bottlenecks in the processes.

At the facility there are a total of 13 centrifuges: seven for thickening WAS and six for dewatering anaerobically digested sludge. Table 19-13 summarizes the historical performance of the centrifuges. At current flow rates, five of the six dewatering centrifuges should have been in operation to manage the biosolids flow; however, as a result of the performance of the centrifuges, all six were in operation.

Table 19-13 – Historical Performance of the Ottawa ROPEC Thickening and Dewatering Centrifuges

(Adapted from Newbigging *et al.*, 2004)

Parameter	Units	Thickening Centrifuges	Dewatering Centrifuges
Make	-	Alfa Laval High Speed, Solid Bowl	Alfa Laval High Speed, Solid Bowl
Model	-	XM706	XM706
Number of Centrifuges	-	7	6
Throughput	L/s	12.6	8.5
Polymer Usage	kg/dry tonne	0.95	10.60
Cake Solids	%	5.9	30.0
Centrate TSS Concentration	mg/L	1104	756

The optimization study for the centrifuges included:

- Baseline testing to determine current performance;
- Review and optimization of polymer usage and dosage;
- Review and optimization of centrifuge operational settings;
- Stress testing of modified centrifuges; and
- Development of a comprehensive computer model that can determine the impact of the optimization study results on the overall system capacity.

WAS Thickening Optimization

Stress testing was performed in order to determine the maximum throughput, optimal operating settings, polymer dosage requirements, and impacts on the thickened/dewatered sludge concentration and centrate quality.

Jar testing was undertaken to determine the optimal polymer dosage for the WAS thickening centrifuges. The jar testing indicated that a dosage of 2 to 4 kg of polymer per tonne dry solids (DS) was able to produce a stronger sludge able to release more free water. As the current dosage of polymer was 0.8 to 1.0 kg/dry tonne, increasing the dosage could improve the thickening performance.

The operational settings of the WAS thickening centrifuge were reviewed and compared to the most recent recommended setting from the manufacturer. The ring dams and the liquid discharge diameters were investigated to determine if increasing the diameter could improve the centrifuge performance. Testing of three centrifuges (numbers 4, 6 and 7) was undertaken by changing the ring dam settings from 342 mm to three different dam settings (354.5 mm for centrifuge 4, 350.5 mm for centrifuge 6, and 352.5 mm for centrifuge 7). Increasing the liquid discharge diameter increases the throughput through the centrifuge. The initial tests showed that, at constant polymer dosage, increasing the throughput decreased the thickened WAS (TWAS) concentration of all the tested centrifuges with the greatest impact on the centrifuge set at a liquid discharge diameter of 354.5 mm (centrifuge 4). For this reason, centrifuge 4 was removed from further testing.

Testing continued by increasing the sludge feed rate and polymer dosage to centrifuges 6 and 7 to determine the impact on the TWAS concentration. Three tests were performed. First, both the feed rate and polymer dosage were increased at the same time to determine the impact on TWAS concentration. It was determined that at a feed rate of 23.7 L/s, the thickened WAS concentration was relatively constant. This feed rate was then used in the subsequent test where it was kept constant (23.7 L/s) and polymer dosage was increased to determine the impact of polymer dose on the thickened WAS TSS concentration as well as the centrate concentration. The polymer dosage was above the historical polymer dosage 0.8 to 1.0 kg/tonne but was limited to 2.1 kg/dry tonne due to limited capacity in the dosing system. The results of increasing the polymer dosage were conflicting in terms of TWAS concentration. The TWAS concentration in centrifuge 6 decreased as the polymer dosage was increased whereas the opposite was evident for centrifuge 7. Increasing the polymer dosage for both centrifuges decreased the suspended solids concentration in the centrate from above 1,500 mg/L to below 1,200 mg/L as the dosage increase from 1.3 to 2.1 kg/t DS.

Overall the results of the optimization study on the WAS thickening centrifuge included:

- WAS centrifuge feed rate of up to 23.7 L/s possible without impacting thickened WAS concentration or centrate quality;

- WAS centrifuges optimal settings for ring dam were between 350.5 mm and 352.5 mm; and

- Increasing the polymer dosage improves the performance especially in terms of centrate quality. However limitations in the polymer dosing system (pump and piping) limited the range of dosages that could be tested.

Dewatering Centrifuge Optimization

The dewatering centrifuges had operated well at influent flows of 8.5 L/s. To gain an understanding of the actual capacity of the centrifuges, stress testing of one dewatering centrifuge was undertaken. The test involved increasing the feed rate and monitoring the TS of the cake and the TSS concentration of the centrate. The feed rate was increased from 9 to 15 L/s during the test. The polymer dosage was increased proportionally to the feed rate to maintain a relatively constant polymer dosage of between 8.8 to 9.4 kg/dry tonne. The results indicated that the cake TS increased from 28 percent at 9 L/s to over 30 percent at 14.2 L/s. The centrate TSS remained fairly constant at approximately 600 mg/L at feed rates of 9 to 14.2 L/s and then spiked at 15 L/s to over 1,000 mg/L. Therefore, the dewatering centrifuge operation could be increased to just below 15 L/s without impacting the performance of the system. The dewatering centrifuges are limited by the capacity of the feed pump. In order to be able to increase the capacity to 15 L/s, installation of new feed pumps will be required.

Conclusion

Once the stress testing was completed, a comprehensive GPS-X™ model was developed to determine the overall impact of the increased capacity of the solids handling processes on overall plant capacity. The model developed was used to determine the expected increase in influent flow and the ability of the solids handling processes to handle the solids generated with one of each of the centrifuges out of service. The thickening centrifuges were shown to be adequate for a plant capacity of 650,000 m^3/d (at ~23 L/s) while the dewatering centrifuges were able to manage the solids produced at a plant capacity in excess of 700,000 m^3/d.

The optimization program also showed that some of the centrifuges could be taken off-line at current flows, saving operating costs until the demand increased.

19.7.2 Effect of MicroSludge™ on Anaerobic Digester and Residuals Dewatering at the Los Angeles County Sanitation Districts' Joint Water Pollution Control Plant (JWPCP)

The following case study is based on information presented in Rabinowitz and Stephenson (2006).

MicroSludge™ is a patented process that liquefies thickened WAS in order to increase the destruction of sludge within mesophilic anaerobic digestion. The process involves a caustic chemical pretreatment step to weaken the membrane of microbial cells followed by a high pressure cell disrupter which liquefies the TWAS. A full scale demonstration study of the process was undertaken at the Los Angeles County JWPCP in California. The objective of the study was to compare, under similar operating conditions, the performance of digesters that receive TWAS and thickened primary sludge processed by the MicroSludge™ process to the performance of digesters that receive untreated TWAS and primary sludge.

The JWPCP treats 1,203,000 m^3/d of sewage as well as the solids from the District's upstream water reclamation facilities. There are a total of 24 mesophilic digesters, each with a volume of 14,500 m^3. The system installed for the trials had two cell disrupters and all equipment needed with a capacity to process 190 m^3/d at 100 percent utilization and a TWAS TS concentration of up to 8 percent. The characteristics of the TWAS and thickened primary sludge feed to both the control and MicroSludge™ fed digesters are outlined in Table 19-14. The digesters feed rate during the study was 8,000 L/h.

Table 19-14 – Thickened WAS and Thickened Primary Sludge Characteristics

(Adapted from Rabinowitz and Stephenson, 2006)

Parameter	Unit	Thickened WAS	Thickened Primary Sludge	Combined TWAS and Thickened Primary Sludge
Percent of total flow to digester	%	25	75	100
Average TS concentration	%	5.4	n/a	4.3
Volatile fraction	%	78	n/a	75
Percentage of dry solids	%	32	68	100
Notes: n/a – not available				

In sludge that had undergone the Microsludge™ process, there was a significant increase in the distribution of TS, VS, BOD and COD, with TWAS being converted to an almost completely liquefied form.

Results of the study indicated that the MicroSludge™ fed digester did not have any foam or filamentous microorganisms present. In the past, there had been problems with severe foaming and filamentous microorganisms which caused issues with digester operation. Table 19-15 outlines the performance of both the MicroSludge™ fed digester and the control digester.

Table 19-15 - MicroSludge™ Digester and Control Digester Performance Data

(Adapted from Rabinowitz and Stephenson, 2006)

Parameter	Unit	MicroSludge™ Fed Digester	Control Digester
VS Reduction	%	54.4 - 58.1	51.4 - 54.4
Total BOD Feed Concentration	mg/L	15,210	15,740
Soluble BOD Feed Concentration	mg/L	4,000	460
Soluble BOD Effluent Concentration	mg/L	115	165
Total COD Feed Concentration	mg/L	47,070	47,170
Soluble COD Feed Concentration	mg/L	5,630	1,020
Soluble COD Effluent Concentration	mg/L	530	485
Methane	% (dry volume basis)	62.3	61.1
Carbon Dioxide	% (dry volume basis)	34.3	35.9
Oxygen	% (dry volume basis)	0.7	0.6
Nitrogen	% (dry volume basis)	2.6	2.0
Biogas Generation per VS Reduced	m^3/kg	0.997	0.994

Overall, there was not a significant difference between the MicroSludge™ fed digester and the control digester with the exception of the concentration of soluble BOD and COD in the influent sludge, with the MicroSludge™ fed digester having considerably higher soluble concentrations. There was some indication that the MicroSludge™ fed digester might not have been fully acclimatized for the first portion of the study which may have skewed the results. There was a temperature increase shown between the MicroSludge™ fed digester and the control digester which resulted in a 17 percent lower steam demand in the MicroSludge™ fed digester. The viscosity of the MicroSludge™ fed sludge was decreased by 90 percent in comparison to the control fed sludge which resulted in more efficient pumping of the sludge.

Following digestion, the dewatering characteristic of the MicroSludge™ digested sludge and control sludge were compared by dewatering the sludge in a belt filter press. There was no noticeable difference in the centrate/filtrate quality in terms of TDS, COD, BOD, TKN, TAN, TP or metal concentrations. The dewatered cakes also had similar metal concentrations.

Overall, the main improvements of the MicroSludge™ fed sludge over the control sludge was that the MicroSludge™ fed digested sludge did not appear to have any filamentous microorganisms which could cause foaming. Also, the former required lower amounts of steam and the MicroSludge™ fed digested sludge could be more efficiently pumped.

19.8 REFERENCES

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CHAPTER 20

IMPACTS OF SLUDGE PROCESSING RECYCLE STREAMS, LEACHATE AND SEPTAGE

CHAPTER 20

IMPACTS OF SLUDGE PROCESSING RECYCLE STREAMS, LEACHATE AND SEPTAGE

20.1 OVERVIEW OF SLUDGE PROCESSING RECYCLE STREAMS

20.1.1 Types of Recycle Streams and Expected Characteristics

Sidestreams are the internal recycle flows produced by processes occurring within the sewage treatment plant. Sidestreams can be produced by liquid treatment processes such as backwashes from tertiary filters, screening or grit washing operations, or waste activated sludge. Sidestreams can also be produced during the stabilization of sludge by biological, chemical or thermal processes and during the concentration of the sludge stream by thickening or dewatering processes. Figure 20-1 illustrates some of the potential sidestreams that can occur in a large, complex STP (Nutt, 1996). The major sidestreams can include:

- grit separator overflow;
- WAS if recycled to the primary clarifiers for co-thickening with the primary sludge;
- sludge thickening process recycles from unit processes such as gravity thickeners, DAF thickeners, thickening centrifuges, gravity belt or drum thickeners;
- sludge digester (aerobic or anaerobic) supernatant;
- thermal conditioning decant liquor;
- sludge dewatering process recycles, either filtrate or centrate, and possibly including washwater;
- scrubber water or other odour or air emission control process recycles; and
- filter backwash from tertiary filtration units.

The liquid treatment process recycle streams generally represent an increase in hydraulic load, but do not significantly increase the contaminant load on the STP under normal operating conditions. Sludge processing recycle streams can contain high concentrations of particulate matter, organic matter, nitrogen and phosphorus that can significantly increase the loading on the plant. The impacts are often compounded by the intermittent nature of these recycle streams. This section of the Guidance Manual focuses specifically on recycle streams from sludge processing.

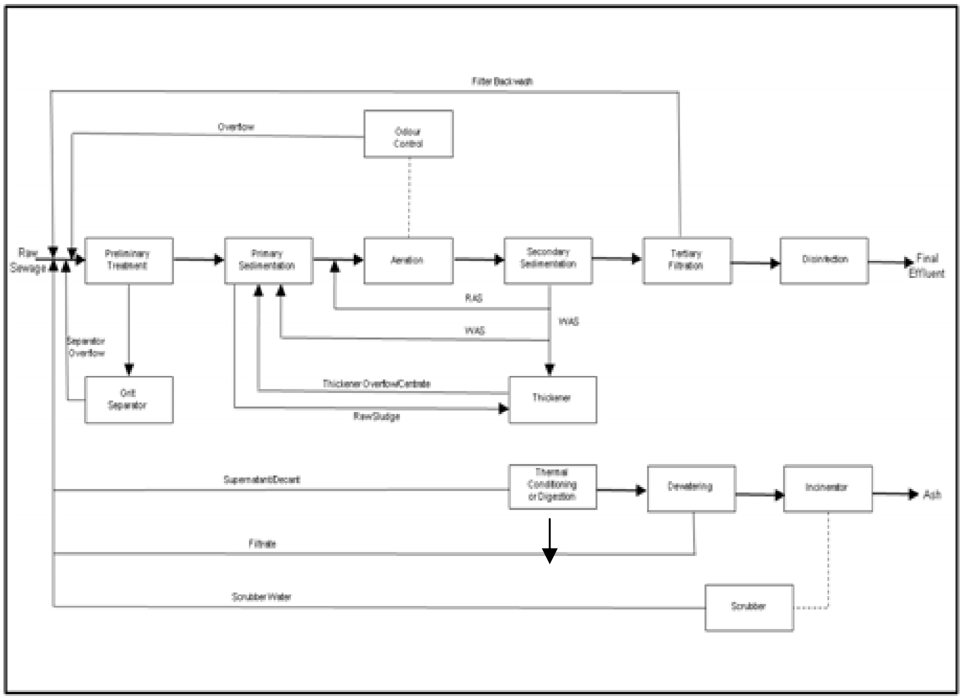

Figure 20-1 – Potential Sidestreams in Sewage Treatment Plants

(Adapted from Nutt, 1996)

Table 20-1 summarizes the BOD_5 and TSS concentrations in typical sludge processing recycle streams (Metcalf & Eddy, 2003). The most common sludge recycle streams in smaller sewage treatment plants are supernatant streams from sludge digestion, either aerobic digestion in smaller extended aeration type plants or anaerobic digestion in larger conventional activated sludge plants. Table 20-2 and Table 20-3 present typical characteristics of supernatant return streams from aerobic and anaerobic digestion processes, respectively (WEF, 1990).

Of particular importance is the high concentration of nitrogenous compounds in these recycle streams. Supernatant from anaerobic digestion processes contain high concentrations of TAN which can have a significant impact on BNR plants or STPs that are required to nitrify as a result of the increased nitrogen load and the intermittent nature of this loading. In mesophilic aerobic digestion processes, the bulk of the nitrogen in the supernatant is present as TKN and associated with the suspended solids or as nitrate as a result of nitrification that occurs in the digester.

Phosphorus in anaerobic or aerobic digester supernatant streams in conventional treatment plants, although present at elevated concentrations, is generally associated with particulate matter and is removed in the settling processes. However, in STPs practicing biological phosphorus removal (Section 16.1.2), the excess phosphorus taken up by the PAOs can be released during anaerobic digestion and recycled to the biological process. In such cases, the recycle of sludge processing streams should therefore be avoided.

Biosolids dewatering process recycle streams (centrate or filtrate) generally will have characteristics similar to the supernatant streams, depending on whether the biosolids are aerobically or anaerobically digested. Concentrations of particulate contaminants may be higher or lower in the dewatering process recycle stream depending on the efficiency of solids capture in the dewatering process.

Table 20-1 - BOD and TSS Content of Selected Sludge Processing Recycle Streams

(Adapted from Metcalf & Eddy, 2003)

Operation	BOD (mg/L)		TSS (mg/L)	
	Range	Typical	Range	Typical
Gravity Thickening Supernatant:				
Primary Sludge	100-400	250	80-350	200
Primary Sludge + Waste Activated Sludge	60-400	300	100-350	250
Flotation Thickening Subnatant	50-1200	250	100-2500	300
Centrifuge Thickening Centrate	170-3,000	1,000	500-3000	1,000
Aerobic Digestion Supernatant	100-1,700	500	100-10,000	3,400
Anaerobic Digestion (Two-stage, High-rate) Supernatant	500-5,000	1000	1000-11,500	4,500
Centrifuge Dewatering Centrate	100-2,000	1000	200-20,000	5,000
Belt-filter Press Filtrate	50-500	300	100-2,000	1,000
Recessed-plate-filter Press Filtrate	50-250	-	50-1,000	-
Sludge Lagoon Supernatant	100-200	-	5-200	-
Sludge Drying Bed Under-drainage	20-500	-	20-500	-
Composting Leachate	-	2000	-	2,000
Incinerator Scrubber Water	30-80	-	600-8000	-

Table 20-2 – Characteristics of Supernatant from Aerobic Digestion Processes

(From WEF, 1990)

Parameter	Range (mg/L)	Typical Value (mg/L)
pH	5.9 - 7.7	7.0
BOD_5	9.0 - 1,700	500
Soluble BOD_5	4.0 - 183	50
COD	288 - 8,140	2,600
TSS	46 - 11,500	3,400
TKN	10 - 400	170
NO_x-N	-	30
TP	19 - 241	100
Soluble Phosphorus	2.5 - 64	25

Table 20-3 - Characteristics of Supernatant from Anaerobic Digestion Processes

(From WEF, 1990)

Parameter	Primary Plants (mg/L)	Trickling Filters (mg/L)	Activated Sludge Plants (mg/L)
TSS	200 - 1,000	500 - 5,000	5,000 - 15,000
BOD_5	500 - 3,000	500 - 5,000	1,000 - 10,000
COD	1,000 - 5,000	2,000 - 10,000	3,000 - 30,000
TAN	300 - 400	400 - 600	500 - 1,000
TP	50 - 200	100 - 300	300 - 1,000

20.1.2 Potential Process Impacts and Mitigation Measures

As illustrated by the data in Tables 20-1, 20-2 and 20-3, sludge treatment process recycle streams can significantly increase the loading to the liquid treatment processes if not properly managed. These sidestreams can increase the organic and nutrient loading on the liquid treatment processes by 5 to 100 percent (WEF, 1990).

Under the best circumstances, these sidestreams will result in higher energy use for aeration, greater sludge production, and higher operating costs. The design of the liquid train treatment processes must consider the potential impact of solids and organics loadings of these sidestreams. If high solids concentrations are returned to the liquid train in these recycle streams, these solids will be removed in the sedimentation processes and returned to the solids treatment processes, potentially overloading them. BNR processes are particularly sensitive to recycle stream impacts since these recycle streams can return the biologically-bound phosphorus to the liquid train processes.

Table 20-4 summarizes some of the potential process impacts of sludge processing recycle streams and possible measures to mitigate the impacts.

Table 20-4 - Potential Impacts of Sludge Treatment Recycle Streams and Possible Mitigation Measures
(Adapted from Metcalf & Eddy, 2003)

Sludge Treatment Process	Process Impact	Mitigation
Sludge Thickening	• Recycle of dispersed, poorly settling, colloidal solids	• Optimize solids capture in thickening process (Chapter 19) • Add coagulant or flocculant to liquid train processes (Chapters 11 and 14) • Separately thicken primary and biological sludges
	• Floating sludge	• Reduce gravity thickener retention time to prevent gas formation
	• Odours	• Reduce gravity thickener retention time to prevent gas formation • Provide odour control process
	• Solids accumulation in bioreactors and clarifiers	• Optimize solids capture in thickening process (Chapter 19) • Return solids-laden recycle stream to primary clarifiers if possible to reduce impact on bioreactor

Table 20-4 - Potential Impacts of Sludge Treatment Recycle Streams and Possible Mitigation Measures (continued)
(Adapted from Metcalf & Eddy, 2003)

Sludge Treatment Process	Process Impact	Mitigation
Sludge Dewatering	• Recycle of dispersed, poorly settling, colloidal solids	• Optimize solids capture in dewatering process (Chapter 19) • Add coagulant or flocculent to liquid train processes (Chapters 11and 14)
	• Solids accumulation in bioreactors and clarifiers	• Optimize solids capture in dewatering process (Chapter 19) • Return solids-laden recycle stream to primary clarifiers if possible to reduce impact on bioreactor
Sludge Stabilization	• High organic load to biological process of the liquid train	• Increase frequency of supernating from digesters to reduce shock load • Increase biomass inventory in liquid train biological process to accommodate load from recycle flow • Provide equalization of recycle flows (Section 20.4.1) • Provide separate recycle stream treatment (Section 20.4.2)
	• High nutrient (N or P) load to biological process of the liquid train	• Increase frequency of supernating from digesters to reduce shock load • Increase biomass inventory in liquid train biological process to accommodate load from recycle flow • Provide equalization of recycle flows (Section 20.4.1) • Provide separate recycle stream treatment (Section 20.4.2)

20.2 OVERVIEW OF LEACHATE AND SEPTAGE

20.2.1 Expected Characteristics

Sewage treatment plants are increasingly being required to treat leachate from municipal landfill sites and/or septage and hauled sewage from private sewage systems. Although not an internal recycle stream, these high strength wastes can create similar problems if not properly managed at the sewage treatment plant.

The characteristics of landfill leachate vary widely depending on the materials that have been disposed in the landfill, the age of the landfill and the design of the leachate collection and containment system. Landfill leachates are often characterized according to the age of the landfill as young leachate (produced from solid waste less than 5 years old), medium leachate (produced from solid waste between 5 and 10 years old), and old leachate (produced from solid waste more than 10 years old). Table 20-5 presents a summary of the average concentrations of conventional contaminants in young, medium and old landfill leachate based on information reported in the literature (Kang *et al.*, 2002; Quasim, 1994; McBean, 1995).

Table 20-5 - Characteristics of Landfill Leachate

Parameter	Young Leachate	Medium Leachate	Old Leachate	Typical Municipal Sewage
TSS (mg/L)	1,438	143	17.2	200
BOD_5 (mg/L)	15,419	2,342	97.5	170
COD (mg/L)	23,421	5,348	1,367	417
TKN (mg/L)	1,416	1,296	567	33
TAN (mg/L)	1,328	1,088	476	20
TP (mg/L)(1)	155	–	–	7

Note:

1. Total phosphorus was not reported for medium and old leachate. The maximum reported value in the literature for young leachate was 155 mg/L.

Table 20-6 presents a summary of average, minimum and maximum concentrations for conventional parameters in septage and compares septage to typical raw sewage concentrations (MOE, 2008). Heavy metals and other trace contaminants are not typically present at elevated concentrations in septage from domestic sources, although mercury and cadmium have been measured at low levels (MOE, 2008). Septage, in terms of characteristics, is similar to poor quality supernatant from an anaerobic digestion process or the recycle from a poorly operated digested sludge dewatering process.

Table 20-6 - Characteristics of Septage
(Adapted from MOE, 2008)

Parameter	Concentration (mg/L)			EPA Mean [1]	Suggested Design Value [1,2]	Typical Municipal Sewage (mg/L)	Ratio of Septage to Sewage
	Avg.	Min.	Max.				
TS	34,100	1,100	130,500	38,800	40,000	720	55:1
TVS	23,100	400	71,400	25,300	25,000	360	69:1
TSS	12,900	300	93,400	13,000	15,000	210	71:1
VSS	9,000	100	51,500	8,700	10,000	160	62:1
BOD_5	6,500	400	78,600	5,000	7,000	190	37:1
COD	31,900	1,500	703,000	42,800	15,000	430	35:1
TKN	600	100	1,100	700	700	40	17:1
TAN	100	5	120	160	150	25	6:1
Total P	200	20	760	250	250	7	36:1
Alkalinity	1,000	500	4,200	-	1,000	90	11:1
FOG [3]	5,600	200	23,400	9,100	8,000	90	89:1
pH	-	1.5	12.6	6.9	6.0	-	-
Linear Alkyl Sulphonate (LAS)	-	110	200	160	150	-	-

Notes:

1. Values expressed in mg/L, except for pH.
2. The data presented in this table were compiled from many sources. The inconsistency of individual data sets results in some skewing of the data and discrepancies when individual parameters are compared. This is taken into account in offering suggested design values.
3. Fats, oils, and grease.

20.2.2 Potential Process Impacts and Mitigation Measures

The process impacts of leachate and septage additions on a sewage treatment plant can be similar to the impacts of sludge processing recycle streams, particularly if the material is hauled to the sewage treatment plant and batch discharged into the headworks of the facility. MOE *Design Guidelines for Sewage Works* (MOE, 2008) provides design guidance to ensure that the process impacts of treating leachate or septage are minimized. A key design consideration is provision for equalization of the load, either through appropriate receiving facilities at the STP (Section 20.4.1) or by discharge of the material into the sewage collection system at an appropriate point upstream of the sewage treatment plant.

Table 20-7 summarizes some of the potential process impacts of septage and landfill leachate on the sewage treatment plant and possible measures to mitigate the impacts.

Table 20-7 - Potential Impacts of Septage and Landfill Leachate Addition to STPs and Possible Mitigation Measures

Waste Type	Process Impact	Mitigation
Septage	• Overloading of grit removal and screening facilities	• Provide screening and grit removal in dedicated septage receiving facility
	• Odours	• Provide odour control process at receiving station
	• Solids accumulation in bioreactors and clarifiers	• Provide equalization of septage flows (Section 20.4.1) • Discharge material into primary clarifiers if possible or directly to sludge treatment process
Landfill Leachate	• Odours	• Provide odour control process at receiving station
	• High organic and/or nutrient load to biological process	• Increase biomass inventory in liquid train biological process to accommodate load from leachate flow • Provide equalization of leachate flows (Section 20.4.1) • Provide separate leachate pre-treatment (Section 20.4.2)

20.3 EVALUATING PROCESS IMPACTS

20.3.1 Monitoring and Recording

Quantities of septage and landfill leachate received at or discharged into a sewage treatment plant should be recorded, both for operational purposes and for billing purposes. Grab samples of individual batches of material hauled to the sewage treatment plant should be collected and analyzed for conventional parameters including BOD_5, TS or TSS, TKN, TAN, TP and pH. In addition, fats, oils and grease (FOG), heavy metals and other parameters that are included in the municipal sewer use by-law should be measured to prevent the illegal discharge of hazardous or industrial waste into the sewage works.

Flows and qualities of all internal recycle streams should be monitored on a routine basis so that the contribution of these streams to the overall liquid train treatment process loading can be determined.

The capability to measure recycle stream flows and to collect samples (ideally composite samples representative of the average composition of the stream) should be provided.

If the internal recycle streams are returned to the plant upstream of the raw sewage composite sampler and flow meter, the individual streams that contribute to the combined stream entering the treatment process should be sampled so that the actual characteristics of the raw sewage entering the works can be determined by mass balance. Monitoring of the sludge treatment process recycle streams will also allow the performance of the sludge treatment processes to be determined (see Chapter 19) and the need for optimization identified.

20.3.2 Solids Mass Balances

Solids mass balances are powerful tools that can be used to assess the impact of sludge processing recycle streams on the liquid train treatment processes and the performance of sludge treatment processes, among other uses.

Typically, poorly operated sludge treatment processes will recycle high concentrations and loads of solids to the liquid treatment train. If a solids mass balance suggests that excessive amounts of primary clarifier sludge and/or WAS are being pumped to the sludge treatment processes, it can be an indication that solids from the sludge treatment processes are being recycled through the system. In such cases, optimization of the sludge treatment process should be undertaken to reduce the solids load on the liquid treatment train (Chapter 19).

Metcalf & Eddy (2003) provides a detailed example of the preparation of a solids mass balance for a secondary treatment plant.

20.4 MITIGATION MEASURES

20.4.1 Equalization of Loading

Many sludge treatment processes, such as thickening or dewatering, do not operate continuously. As a result, recycle streams from these processes can significantly increase the loading to the liquid treatment train if the weekly (168 hours) sludge production is processed over two to four days, six hours per day (12 to 24 hours of processing) and the recycle streams returned directly to the liquid train. Similarly, aerobic and anaerobic digestion or biosolids holding tanks are often manually decanted once or twice per day for a few hours, producing a slug load on the liquid treatment train. High strength landfill leachate or septage hauled into a sewage treatment plant and batch discharged directly into the liquid treatment train will have a similar impact.

Table 20-8 illustrates the impact of supernating an anaerobic digester two hours per day in the morning and evening on raw sewage quality, based on a simple mass balance (Nutt, 1996). During the period when supernatant is returned to the plant, the average loading increase for all parameters is about 85 percent while nutrient loads (N and P) can almost double. The increase in loading can result in deterioration in plant performance due to increased oxygen demand, increased demand for phosphorus precipitant, and a spike in the nitrogen load to the bioreactor of the secondary treatment process.

Table 20-8 - Effect of Digester Supernatant on Raw Sewage Quality

Parameter	Concentration (mg/L)			Loading Increase (%)
	Supernatant	Raw Sewage	Plant Influent	
TSS	5,000	200	367	84
VSS	3,100	150	252	68
Total BOD_5	2,700	200	287	44
Soluble BOD_5	1,200	65	104	60
Total COD	7,300	270	514	90
Soluble COD	2,300	130	205	58
Total P	190	7.0	13.4	91
Soluble P	150	5.5	10.5	91
Total N	1,000	30.0	63.7	112
Soluble N	850	22.5	51.3	126

Load equalization or continuous discharge of sidestreams and concentrated waste streams such as landfill leachate or septage will reduce the impact of these streams significantly. In the example provided in Table 20-8, return of the supernatant stream 24-hours per day would reduce the average loading increase from about 85 percent to less than 10 percent. Load equalization can be provided by:

- Extending the hours of operation of the source process;
- Discharging hauled waste or landfill leachate into the sewage collection system upstream of the STP; or
- Providing an equalization storage tank for the concentrated waste stream and the capability to pump the concentrated waste over an extended period and in proportion to the plant inflow (or during reduced flow/load periods).

20.4.2 Sidestream Treatment

Separate treatment of sidestreams has generally only been implemented in instances where stringent effluent nitrogen limits apply and the sidestream contributes a significant nitrogen load to the liquid treatment train. Typical instances are the recycle of centrate or filtrate from dewatering of anaerobically digested biosolids. Although this stream may represent as little as 1 percent of the plant flow, it can contribute 20 to 40 percent of the plant nitrogen load (Stinson, 2007). Issues related to elevated solids and BOD_5 loadings can typically be dealt with by optimization of the sludge treatment process or by equalization.

A number of novel centrate/filtrate treatment processes specifically designed to remove nitrogen have been developed that take advantage of some of the unique characteristics of this recycle stream such as its elevated temperature, low alkalinity and relatively low organic content. This includes both biological and physical-chemical processes. Some of these processes are still at the developmental stage while others have been implemented at demonstration-scale or full-scale. None of these processes are currently being used in Ontario. Table 20-9 summarizes some of the treatment processes that have been applied or considered for sidestream treatment of centrate and filtrate from dewatering of anaerobically digested sludge. Stinson (2007) provides additional information on these technologies.

A significant benefit of some of the biological treatment options to control nitrogen in these sidestreams is the potential for bioaugmentation. The return of the biologically treated sidestream to the mainstream nitrification process can seed the bioreactors with a continuous stream of nitrifying bacteria. This bioaugmentation will improve the stability of the mainstream treatment process and allow operation at lower SRT than would normally be possible (Parker and Wanner, 2007).

Table 20-9 - Novel Treatment Options for Dewatering Centrate and Filtrate
(Adapted from Stinson, 2007)

Biological Processes	Physical-Chemical Processes
Nitrification/Denitrification • Inexpensive Nitrification (In-Nitri) Process • New York City Department of Environmental Protection Aeration Tank #3 (AT#3) Process • Bio-Augmentation Batch Enhanced (BABE) Process • Mainstream Autotrophic Recycle Enabling Enhanced N-removal (MAUREEN) Process	Ammonia Stripping • Steam • Hot Air
Nitritation/Denitritation • Single reactor system for High Activity Ammonia Removal Over Nitrite (SHARON) Process	Ion Exchange • Ammonia Recovery Process (ARP)
De-ammonification • Strass Process • Anaerobic Ammonium Oxidation (ANAMMOX) Process • Combined SHARON/ANAMMOX Process	Struvite (Ammonium Magnesium Phosphate, MAP) Precipitation

20.4.3 Other Measures

As noted above, extending the operating time and frequency of operation of the process producing the sidestream and optimizing the sludge treatment process can reduce the impact of sidestreams on the liquid treatment train.

Other measures that can be used to mitigate the impacts of recycle stream, leachate or septage include:

- Returning the sidestream to an appropriate point in the liquid treatment train. Sidestreams should be returned to a point in the process that will ensure effective treatment and prevent operational and maintenance problems. For example, recycle streams that can contain elevated suspended solids concentrations should be returned upstream of primary clarifiers if possible to reduce the solids loading and accumulation in the secondary treatment process;
- Provide the capability to feed the recycle or concentrated waste stream in proportion to raw sewage flow. The recycle or concentrated waste streams can be used to maintain a more consistent loading to the process if returned during periods of low flow and load during the early morning hours;

- Optimize the secondary treatment process SRT to accommodate the increased contaminant loading (Chapter 12); and
- Adding alkalinity in low pH and low alkalinity environments, as low pH and alkalinity inhibits nitrification (Chapter 12).

20.5 CASE HISTORIES

The following case histories are based on information presented in Briggs *et al.* (2001) for two large sewage treatment plants, the Lakeview WPCP and an unidentified CAS plant referred to as "Plant A", with complex sludge management processes.

20.5.1 Lakeview WPCP

The Lakeview WPCP, at the time of the study, was a 336,000 m^3/d conventional activated sludge plant with WAS thickening by centrifuges, thermal conditioning, anaerobic treatment of the thermal conditioning liquor, sludge dewatering on vacuum coil filters, and thermal oxidation in fluid bed incinerators. The Lakeview WPCP also receives and treats sludge from a neighbouring STP. A process schematic of the facility in Figure 20-2 shows the major recycle streams including:

- WAS thickening centrate;
- anaerobically treated thermal conditioning liquor; and
- vacuum filter filtrate and washwater.

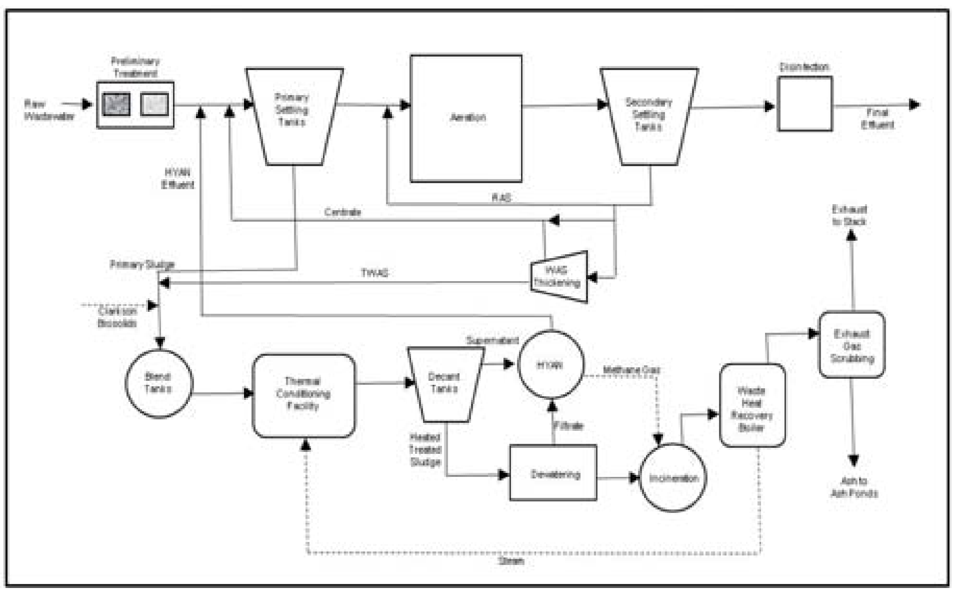

Figure 20-2 - Simplified Process Schematic of Lakeview WPCP

(Adapted from Briggs *et al*., 2001)

The recycle streams were determined to contribute about 50 percent of the TSS loading and about 33 percent of the BOD_5 loading to the plant. These recycle streams significantly impacted on plant capacity and the estimated capital costs associated with a plant expansion.

Process improvements to reduce recycle stream loading included:

- optimization of the WAS thickening centrifuges through polymer addition to increase solids capture efficiency and capacity; and
- separate handling of the sludges from the neighbouring STP.

Process simulation modelling suggested that the reduction in loading from the recycle streams would result in an increase in plant capacity of about 54,000 m^3/d or about 15 percent. It was estimated that the total capital cost to optimize WAS thickening and separate handling of the other plant sludges would cost about $222/m^3 of capacity gained compared to plant expansion cost of about $500/m^3 to provide the same capacity.

20.5.2 Unidentified CAS Plant ("Plant A")

Plant A is a conventional activated sludge plant with a rated capacity of 455,000 m^3/d with gravity thickening of primary sludge, dissolved air flotation (DAF) thickening of WAS, anaerobic digestion, sludge dewatering on vacuum filters and thermal oxidation in multiple hearth incinerators. A process schematic of the facility is shown in Figure 20-3.

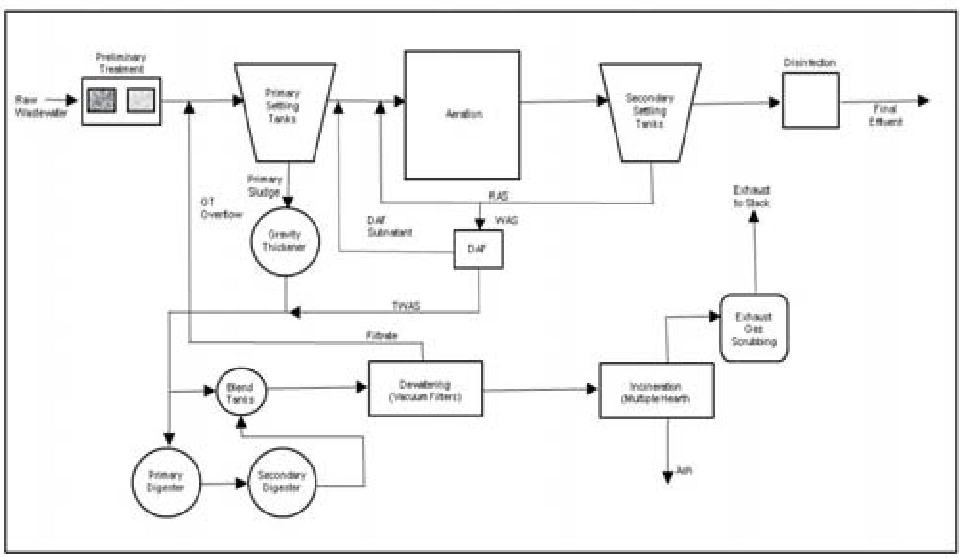

Figure 20-3 – Simplified Process Schematic of Plant A

(Adapted from Briggs *et al.*, 2001)

Major recycle streams at Plant A included:

- supernatant from primary sludge gravity thickeners;
- subnatant from WAS DAF thickeners;
- filtrate from digested sludge vacuum filters; and
- ash lagoon supernatant.

In this case, it was found that the recycle streams represented more than 50 percent of the solids loading to the plant. Improvements to the sludge management processes, including replacing the WAS DAF thickeners with centrifuges and optimizing the operation of the gravity thickener and dewatering process, were estimated to reduce the contribution of solids in the recycle stream by more than 50 percent.

Process simulation modelling was used to predict the improvements in effluent quality that could be achieved by the reduced recycle stream solids loadings. The findings are summarized in Table 20-10. The improvements were predicted to result in an 18 percent reduction in effluent TSS concentrations based on a 30-day average and a 22 percent improvement based on a 7-day average. Similar improvements were predicted in the effluent BOD_5 concentration due to the reduced effluent TSS concentrations.

Table 20-10 – Impact of Reduced Recycle Loadings on Effluent Quality at Plant A
(From Briggs *et al*., 2001)

Condition	Effluent TSS		Effluent BOD_5	
	Max. 7-day Average (mg/L)	Max. 30-day Average (mg/L)	Max. 7-day Average (mg/L)	Max. 30-day Average (mg/L)
Existing	42	34	21	17
Reduced Recycle Loadings	33	28	17	14

20.6 REFERENCES

Briggs, T., S. Nutt, D. Ross and I. Prasad (2001). "Controlling Performance and Capacity Through Sidestream Management". *Proceedings of the 30th Annual WEAO Technical Symposium*, Hamilton, Ontario.

Kang, K.-H., H.S. Shin and H. Park (2002). "Characterization of Humic Substances Present in Landfill Leachates with Different Landfill Ages and its Implications", *Water Research*, vol. 36, pp. 4023-4032.

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WEF (1990). *Operation of Municipal Wastewater Treatment Plants, Manual of Practice No. 11*. ISBN 0-943244-69-2.

CHAPTER 21

REPORTING OF RESULTS

CHAPTER 21

REPORTING OF RESULTS

21.1 FINAL REPORT

The outcome of an optimization study should be a comprehensive report that concisely presents:

- the project background and the rationale for the optimization study;
- the project objectives;
- a concise description of the STP including a summary of the design flows and loadings, the process design of key unit processes (refer to Table 5-1), a process flow diagram (refer to Figure 5-2), and a summary of the C of A treatment and effluent requirements that must be met;
- a summary of key information sources used during the investigation (e.g. historic data, preliminary design reports, Cs of A, MOE inspection reports, annual reports, etc.);
- a summary of historic operating conditions and treatment performance for a period of three to five years (refer to Section 5.2.2);
- a desk-top analysis of the capacity and capability of each major unit process (refer to Section 5.2.3);
- the methodology and findings of all field investigations such as stress tests, tracer tests, oxygen transfer tests, jar tests, etc.;
- an analysis of options to address capacity or performance limitations or increase rated capacity to meet the project objectives;
- the conclusions of the study; and
- any recommendations for follow-up investigations or implementation of improvement measures based on the findings.

Table 21-1 presents a suggested Table of Contents for a STP Optimization Final Report. The level of detail included in the final report should be consistent with the project objectives and the target audience. For example, if an objective of the optimization study is to support an application for a new C of A to re-rate the plant capacity, sufficient detail must be provided in the report for the MOE review engineer to confirm that the proposed changes will consistently and reliably meet the C of A effluent requirements at the re-rated capacity.

21.2 INTERIM REPORTS - TECHNICAL MEMORANDA

Preparation of Technical Memoranda after completion of key activities are an effective means of ensuring that all participants in the optimization study (owner, operations staff, consulting team, etc.) are kept informed of project progress, have an opportunity to review and understand project findings at an early stage, and provide input to the overall project direction.

Table 21-1 – Example Table of Contents for STP Optimization Report

Item	Content
Executive Summary	A concise (2 to 3 page) summary of the project objectives, key findings, conclusions and recommendations.
Table of Contents	Identifies key sections and subsections by title and includes a list of tables, figures and appendices.
Introduction and Background	Provides the rationale for the study and any background information relevant to understanding the need for the process optimization. Should include a list of key information sources used in the study.
Project Objectives	Concisely states the key objective(s) of the study.
Plant Description	Provides a process flow diagram of the STP including key recycle streams and the inter-relationship between liquid and sludge treatment processes. Key design parameters (e.g. ADF, peak flow, effluent requirements, etc.) and sizing of key unit processes/mechanical equipment should be provided.
Historic Data Review	A review of key operating and performance information for a period of three to five years, including flows, raw wastewater characteristics, effluent quality, quality of intermediate streams where available and applicable (e.g. primary effluent quality, WAS and RAS flows and concentrations, raw sludge flows and concentrations, digested sludge flows and concentrations, etc.), and critical operating parameters (e.g. surface overflow rates, solids loading rates, biological system operating conditions such as MLSS, MLVSS, F/M, SRT, chemical dosages, etc.).
Desk-top Capacity Assessment	Results of the desk-top analysis of the capacity of each major unit process under study based on the historic data and comparison to typical design standards and guidelines, including a process capacity chart (see Figure 5-3).
Results of Field Investigations	Methodology and findings of any field investigations undertaken to confirm the capacity assessment or determine the optimum approach to achieve the project objectives.
Assessment of Options	Identification and evaluation of alternative approaches to optimize the plant to meet the project objectives, including both operational and design changes. Should include consideration of constructability, integration with existing system, capital and operating costs, risks, complexity, etc.
Conclusions	Concise summary of the key findings.
Recommendations	Recommendations for implementation of the conclusions or for further investigations.
References	Listing of key reference material.
Appendices	Contains all supporting documentation such as Cs of A, modelling outputs, data from field investigations, details of cost analyses if any, etc.

Each Technical Memorandum (TM) should include:

- an introduction describing the overall objective of the project;
- the specific objective of the TM and how it relates to the overall project;
- a discussion of the methodology, approach and key sources of information used;
- the results of the specific activity described in the TM; and
- conclusions and recommendations.

Relevant data (e.g. modelling and simulation results, tracer test results, stress test results, etc.) should be appended to the TM.

Table 21-2 presents some possible Technical Memoranda that could be prepared during a comprehensive STP optimization project. TMs prepared to describe the findings of field investigations will depend on the specific field investigations undertaken. These Technical Memoranda should be issued as drafts for review by the project team responsible for the optimization study. Comments on the TM should be compiled and the TM appropriately revised and issued as final. These Technical Memoranda can be incorporated into the final report.

Table 21-2 – Possible Technical Memoranda

TM Title	TM Contents
#1: Existing Conditions	• Description of STP • Historic Data Review • Desk-top Capacity Assessment
#2: Field Investigations Work Plan	• Outlines Work Plan for suggested field investigations based on findings of TM#1 • Provides detailed description of test methodology, sampling and analytical requirements, process loadings and treatment targets, operations staff support requirements, notification requirements, and any health and safety considerations
#3: Oxygen Transfer Testing	• Methodology, results and conclusions of oxygen transfer testing to determine oxygen transfer efficiency and capacity
#4: Process Modelling and Simulation	• Methodology used to calibrate and verify the simulation model, conditions modelled, and the model outcomes
#5: Clarifier Testing	• Methods used for dye testing and stress testing of secondary clarifiers and test findings
#6: Options to Optimize Plant Performance	• Description of options being considered • Criteria considered in the assessment • Evaluation of each option against the criteria • Selection of preferred option and justification for selection

21.3 WORKSHOPS

Workshops can be an effective means of communicating findings to all project participants, including plant administration, management and operations staff, and regulators, at key points in the project and for soliciting input on key decisions.

The objectives and desired outcomes of the Workshop must be clearly communicated to the participants. Important technical information that will be discussed at the Workshop should be provided to the participants in advance to ensure that informed feedback and input can be obtained.

Key points in the project where Workshops can prove useful are:

- at project initiation, to introduce the project participants, review project background and objectives, and provide a brief overview of the work plan to be executed and the project schedule;
- after completion of the historic data review, to present the findings of the desk-top analysis, identify process or capacity limitations, and discuss the proposed field investigations; and
- after the analysis of options, to present the findings of the field investigations, proposed solutions to achieve the project objectives, the evaluation of options and to obtain input to the selection of the preferred option(s).

Additional workshops may be beneficial depending on the study scope and duration. Workshops should not replace regular project meetings with plant management and operations staff to discuss specific activities, particularly field investigations.

Workshop notes should be compiled and included as an Appendix to the final project report.

21.4 IMPLEMENTATION OF RECOMMENDATIONS AND FOLLOW-UP

The time required to implement the recommendations from the optimization study will depend on the nature of the recommendations. Operational changes such as increasing bioreactor SRT, increasing chemical dosage, or modifying sludge pumping rates can be implemented quickly by operations staff. Design changes such as installing baffles in clarifiers, retrofitting to fine bubble aeration to improve oxygen transfer efficiency, or changing chemical dosage points can be more time-consuming, likely requiring a detailed design phase, C of A amendment, tendering and construction.

Regardless of the nature of the upgrade, it is important to ensure that there is follow-up monitoring to determine how effective the recommended upgrade was in achieving the original optimization objective. If performance enhancement was the primary objective, a post-implementation monitoring program should be undertaken to compare the performance of the unit process after implementation with the performance achieved prior to implementation. If cost reduction or energy efficiency was the primary objective, comparative operating cost or energy use data, pre- and post-implementation, should be collected.

Documentation of the success of the optimization project is critical to ensure on-going support from management for further optimization activities.

APPENDICES

APPENDIX A

LIST OF ACRONYMS

Acronym	Definition
1-D	One-dimensional
2-D	Two-dimensional
3-D	Three-dimensional
ADF	Average Daily Flow
ASCE	American Society of Civil Engineers
ATAD	Autothermal Thermophilic Aerobic Digestion
AV	Area-Velocity
AWTF	Advanced Water Treatment Facility
BAF	Biological Aerated Filter
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
BOD_5	5-day Biochemical Oxygen Demand
BPR	Biological Phosphorus Removal
CAS	Conventional Activated Sludge
$CBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
CCP	Composite Correction Program
CEPA	*Canadian Environmental Protection Act*
CEPT	Chemically Enhanced Primary Treatment
CFD	Computational Fluid Dynamics
CFU	Colony Forming Units

Acronym	Definition
CO_2	Carbon Dioxide
COD	Chemical Oxygen Demand
C of A	Certificate of Approval
CPE	Comprehensive Performance Evaluation
CSO	Combined Sewer Overflow
CTA	Comprehensive Technical Assistance
DAF	Dissolved Air Flotation
DND	Department of National Defence
DO	Dissolved Oxygen
DS	Dry Solids
DSS	Dispersed Suspended Solids
DWQMS	Drinking Water Quality Management Standard
EAAB	Environmental Assessment and Approvals Branch
EGL	Energy Grade Line
EMS	Environmental Management Systems
EPA	Environmental Protection Agency
EPS	Extracellular Polymeric Substance
ESS	Effluent Suspended Solids
F/M_v	Food-to-Microorganism Ratio
FEDWA	Flocculating Energy Dissipating Well Arrangements
FOTE	Field Oxygen Transfer Efficiency
FSS	Flocculated Suspended Solids

Acronym	Definition
GBT	Gravity Belt Thickener
GIS	Geographical Information Systems
HGL	Hydraulic Grade Line
HHRAP	Hamilton Harbour Remedial Action Plan
HRT	Hydraulic Residence Time
IFAS	Integrated Fixed-film Activated Sludge
I/I	Infiltration / Inflow
ISF	Intermittent Sand Filter
ITA	Instrument Testing Association
JWPCP	Joint Water Pollution Control Plant
MBBR	Moving Bed Biofilm Reactor
MBR	Membrane Bioreactor
MCRT	Mean Cell Residence Time
MF	Microfiltration
MGD	Million Gallons Per Day (U.S.)
MLE	Modified Ludzack-Ettinger
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
MOE	Ontario Ministry of the Environment (ministry)
MOEE	Ministry of Environment and Energy
MSDS	Material Safety Data Sheet
N	Nitrogen

Acronym	Definition
N_2	Nitrogen gas
NO_2^-	Nitrite
NO_3^-	Nitrate
NTU	Nephelometric Turbidity Units
O_2	Molecular Oxygen
O&M	Operations and Maintenance
ORP	Oxidation-Reduction Potential
OTE	Oxygen Transfer Efficiency
OTR	Oxygen Transfer Rate
P	Phosphorus
PA	Process Audit
PAO	Phosphate Accumulating Organism
PCP	Pollution Control Plant
PFD	Process Flow Diagram
PFU	Plaque-Forming Units
P&ID	Process and Instrumentation Diagram
PLC	Programmable Logic Controller
QA	Quality Assurance
QMS	Quality Management System
RAP	Remedial Action Plan
RAS	Return Activated Sludge
RBC	Rotating Biological Contactor

Acronym	Definition
RDT	Rotary Drum Thickener
ROPEC	Robert O. Pickard Environmental Centre
SAE	Standard Aeration Efficiency
SAGR	Submerged Attached Growth Reactor
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
SCC	Standards Council of Canada
SDB	Standards Development Branch
SLR	Solids Loading Rate
SOP	Standard Operating Procedure
SOR	Surface Overflow Rate
SOTE	Standard Oxygen Transfer Efficiency
SOTR	Standard Oxygen Transfer Rate
SOUR	Specific Oxygen Uptake Rate
SRT	Solids Retention Time
SSV	Settled Sludge Volume
SSVI	Stirred Sludge Volume Index
STP	Sewage Treatment Plant
STPOP	Sewage Treatment Plant Optimization Program
SVI	Sludge Volume Index
SWD	Side Water Depth
TAN	Total Ammonia-Nitrogen

Acronym	Definition
TDH	Total Dynamic Head
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMP	Transmembrane Pressure
TN	Total Nitrogen
TP	Total Phosphorus
TPAD	Temperature Phased Anaerobic Digestion
TRC	Total Residual Chlorine
TS	Total Solids
TSS	Total Suspended Solids
TVS	Total Volatile Solids
TWAS	Thickened Waste Activated Sludge
UF	Ultrafiltration
USGPM	U.S. Gallons per Minute
UV	Ultraviolet
UVT	Ultraviolet Transmittance
VE	Value Engineering
VFA	Volatile Fatty Acid
VFD	Variable Frequency Drive
VS	Volatile Solids
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge

Acronym	Definition
WEAO	Water Environment Association of Ontario
WEF	Water Environment Federation
WERF	Water Environment Research Foundation
WHMIS	Workplace Hazardous Materials Information System
WPCC	Water Pollution Control Centre
WPCP	Water Pollution Control Plant
WRF	Water Reclamation Facility
WTC	Wastewater Technology Centre
WWTP	Wastewater Treatment Plant

APPENDIX B

Sewage Treatment Plant
SELF ASSESSMENT REPORT

PLANT NAME:

CONTENTS

SELF ASSESSMENT REPORT SUMMARY:

PLANT NAME: ____

WORKS NUMBER:

Section Number in Self Assessment Report			Actual Value	Concern Level	Action Level	Maximum Possible
1. Effluent Compliance Plant Performance Evaluation	C1.2 / C1.3 C1.4	Pg 15	☞ ☞	60 20	60 30	⊙ 60 ⊙ 90
2. Plant Capacity - Existing - 5 Year Projection	C2.1 C2.2	Pg 17 pg 18	☞ ☞	20 10	30 30	⊙ 90 ⊙ 90
3. Combined Sewer Overflows / Plant Bypasses	C3.2, 3.3 & 3.4	Pg 19/20	☞	40	60	⊙ 90
4. Sludge Handling - Storage/Disposal - Sludge Accountability	C4.2, 4.3 & 4.4 C4.5	Pg 21 pg 22	☞ ☞	20 15	30 20	⊙ 50 ⊙ 50
5. Effluent Sampling / Analysis	C5.1 & 5.2	Pg 26	☞	25	-	⊙ 50
6. Equipment Maintenance	C6.1 & 6.2	Pg 28	☞	25	-	⊙ 50
7. Operator Training / Certification	C7.3, 7.4 & 7.5	Pg 29	☞	20	-	⊙ 50
8. Financial Status	C 8.1 & 8.2	Pg 30	☞	30	-	⊙ 50
Total Point Score for your facility =			☞			⊙ 720

REPORT POINT RANGE:

[ ] 0 - 149 points

VOLUNTARY RANGE - No major deficiency has been identified in this facility. Owner may evaluate and implement steps to address minor problems identified in the report voluntarily.

[ ] 150 - 199 points

RECOMMENDED RANGE - Some deficiencies have been identified (at Concern Level) in this facility. Owner is recommended to implement steps to address the identified problems. If the problems are complicated in nature, a Comprehensive Performance Evaluation (CPE) study is recommended.

[ ] 200 - 720 points OR
THE SCORE OF ANY SECTION IN THE REPORT EQUALS TO OR EXCEEDS THE VALUE GIVEN IN THE "ACTION LEVEL" COLUMN ON THE TABLE ABOVE.

ACTION RANGE - Some major performance limiting factors have been identified (at Action Level) in this facility. These factors directly affect the plant's capability to achieve and maintain compliance in the future. The Owner is required to submit a Remedial Action Work Plan to address the identified problem(s). A Comprehensive Performance Evaluation (CPE) study is an acceptable alternative to a Remedial Action Work Plan.

COMMENTS BY OPERATING AUTHORITY / OWNER / MUNICIPALITY

Referring to the Self Assessment Report Summary on page 2, is there any "Actual Value" scored that equals to or exceeds the value given in the "Action Level" column?

If the answer is YES, you may provide an explanation for this situation in the space below, such as cause, actions proposed, actions taken, etc.:

Comments Provided by:

SECTION A - PLANT INFORMATION

Please provide or verify the following information, and correct if necessary:

Plant Name : ______________________ Certificate of Approval No.:

Works Number : ______________________ ______________________

Facility Classification : ______________________ Date Issued:

____Ministry Region : ______________________ C of A Amendment No.:

District : ______________________ ______________________

Municipality : ______________________ Date Issued: ______________________

Treatment Process Description (refer to Appendix 1 - SA Report Guide page 32 for treatment type descriptions):

______________________ (Major Process Description and Code)

______________________ (Additional Treatment Description and Code)

______________________ (Discharge Mode Description and Code)

______________________ (Disinfection Practice Description and Code)

______________________ (Phosphorus Removal Description and Code)

Design Capacity : ______________________ 1,000 m^3/day

Design Population : ______________________

Population Served : ______________________

Watercourse : ______________________ (Immediate discharge point)

Minor Basin : ______________________

Major Basin : ______________________

UTM Coordinates : Easting ______________________

Northing ______________________

Zone ______________________

Plant Address : ______________________ Phone No. : ______________________

(Physical location) ______________________ FAX No. : ______________________

______________________ Postal Code : ______________________

OWNER INFORMATION

Owner's Name : ______________________

Mailing Address : ______________________ Phone No. : ______________________

______________________ FAX No. : ______________________

______________________ Postal Code : ______________________

OPERATOR INFORMATION

Name of Operating Authority: ______________________

Name of Superintendent or Chief Operator: ______________________

Mailing Address : ______________________ Phone No. : ______________________

______________________ FAX No. : ______________________

______________________ Postal Code : ______________________

SECTION B - DATA SUBMISSION

This Section should be completed and submitted to MOE quarterly.

Plant Name: ______________________________

Municipality: ______________________ **Works Number:** __________

INSTRUCTION FOR QUARTERLY DATA SUBMISSION:

1. Complete and/or update Section B of this report.
2. Photo-copy the completed pages _5_ to _13_ of this Section.
3. Sign at the bottom of this page (on the photo-copy).
4. Submit the photo-copy pages to your District Office of the Ministry of the Environment, **no later than forty-five (45) days after the end of each quarter**.

THIS QUARTERLY DATA SUBMISSION IS FOR (check one):

[ ] 1st Quarter
[ ] 2nd Quarter
[ ] 3rd Quarter
[ ] 4th Quarter

THIS QUARTERLY DATA REPORT WAS COMPLETED BY, ON BEHALF OF THE OPERATING AUTHORITY:

Signature : ______________________

Name : ______________________

Title : ______________________

Contact Phone No. : ______________________

Date : ______________________

B 1 INFLUENT FLOW / CONCENTRATION / LOADING

B1.1 List the maximum, minimum, average daily flows (ADF) and raw sewage concentration received at your facility for the reporting year:

Month	Daily Plant Flow, 1,000 m^3/d			Monthly Average Influent Concentration, mg/L				
	Maximum	Minimum	Average	BOD_5	TSS	TP	TAN	TKN
January								
February								
March								
April								
May								
June								
July								
August								
September								
October								
November								
December								
Annual Average								

B1.2 Calculate the plant Influent loadings based on monthly average daily flows (ADF) and raw sewage concentration received at your facility for the reporting year:

Month	Monthly Average Influent Loading, kg/d = Average Daily Flow (1000 m^3/d) X Influent Concentration (mg/L)				
	BOD_5	TSS	TP	TAN	TKN
January					
February					
March					
April					
May					
June					
July					
August					
September					
October					
November					
December					
Annual Average					

Comments Area - B1 Influent Flow / Concentration / Loading

B 2 EFFLUENT CONCENTRATION / LOADING

B2.1 Provide the monthly average effluent quality data for your facility for the reporting year:

Month	Final Effluent Concentration (Monthly Average)										
	$CBOD_5$ mg/L	TSS mg/L	TP mg/L	TAN mg/L	TKN mg/L	NO_3-N mg/L	NO_2-N mg/L	pH	temp C	R.Cl_2 mg/L	E.Coli /100ml
January											
February											
March											
April											
May											
June											
July											
August											
September											
October											
November											
December											
Annual Average											

B2.2 If your facility has a seasonal discharge, please provide the following discharge information:

Discharge Period From - To (dd/mm/yy)	Discharge Duration (hours)	Volume of Discharge (1,000 m^3)
/ / - / /		
/ / - / /		
/ / - / /		
Annual Total		

B2.3 Calculate and report below the effluent $CBOD_5$, TSS, TP, TAN and TKN loadings based on average monthly flow and effluent concentration data as provided in B2.1 above.

Month	Effluent Loadings (kg/d) = Average Daily Flow (1,000 m^3/d) X Effluent Concentration (mg/L)				
	BOD_5	SS	TP	NH_3-N	TKN
January					
February					
March					
April					
May					
June					
July					
August					
September					
October					
November					
December Average					

B2.4 If your facility has a seasonal discharge, please provide the following discharge loading information:

Discharge Period From - To (dd/mm/yy)	Effluent Load (kg) Per Discharge Period					Loading Discharge Rate (kg/d) Per Discharge Period				
	$CBOD_5$	TSS	TP	TAN	TKN	$CBOD_5$	TSS	TP	TAN	TKN
/ / - / /										
/ / - / /										
/ / - / /										
/ / - / /										
Annual Total						Maximum Discharge Rate (kg/d)				

Effluent Load (kg) = Volume of Discharge (1,000 m^3) X Effluent Concentration (mg/L)

Loading Discharge Rate (kg/d) = Effluent Load (kg) ÷ No. of Discharge Days

Comments Area - B2 Effluent Concentration / Loading

Comments Area - B2 Effluent Concentration / Loading

B 3 COMBINED SEWER OVERFLOWS (CSOs) / PLANT BYPASSES

If you are not responsible for the sewage collection system, please consult the appropriate authority in your municipality to provide information on this Section.

B3.1 Provide the following information for each bypass and/or overflow that occurred within the sewerage system and discharged directly into surface water (excluding overflows with CSO treatment such as storage tanks) for the reporting year. Estimates for duration and volume are acceptable. Attach a separate sheet if more space is needed.

Date (dd/mm/yy)	Location	Start Time	Duration (Hours)	Volume ($1,000 m^3$)	Reason (Refer to Section B3.2 for Codes)	Treatment Provided

B3.2 Provide the following information for each bypass that occurred within the boundary of the sewage treatment plant for the reporting year. Estimates are acceptable for the duration and volume data. **Each day start with a new line**. Photo-copy this page if additional space is required.

Date (dd/mm/yy)	Location	Type P/S	Start Time	Duration hours	Volume $1,000m^3$	Disinfect Y/N/U	Reason Code	Sample Results			
								BOD_5 mg/L	TSS mg/L	TP mg/L	E.Coli /100ml

P = Primary
S = Secondary

Y = Yes
N = No
U = Unknown

Reason Codes
1 = Heavy Precipitation
2 = Snow Melt
3 = Equipment Failure
4 = Eq. Maintenance
5 = Sewer Problems
6 = Power Failure
7 = Exceed Design Capacity
0 = Others

Primary Bypass - the discharge of raw sewage subject to no treatment except grit removal and/or chlorination.

Secondary Bypass - the discharge of sewage that has undergone solids removal at the primary clarifiers but bypassed the secondary treatment process.

B3.3 Plant Bypass Monthly Summary:

Month	Primary Bypass			Secondary Bypass		
	No. of Days (days)	Duration (hours)	Volume (1,000 m^3)	No. of Days (days)	Duration (hours)	Volume (1,000 m^3)
JAN						
FEB						
MAR						
APR						
MAY						
JUN						
JUL						
AUG						
SEP						
OCT						
NOV						
DEC						
TOTAL						
VOLUME OF BYPASS AS % OF * AVERAGE DAILY FLOW (ADF)			%			%

ADF = __________ (1,000 m^3/d)

* % = Volume of Bypass ÷ ADF ÷ 365 × 100

Comments Area - B3 CSOs and Plant Bypasses

B4 SLUDGE QUALITY

B4.1 Provide biosolids analysis data for quarterly samples if biosolids are utilized on agricultural lands. Otherwise, report yearly sample results and enter data on the Quarter Column that corresponds to the date of sampling. *Please note that all metal results are in micro-gram per gram, ug/gm.*

Parameter		First Quarter	Second Quarter	Third Quarter	Fourth Quarter
pH					
Ammonia plus Ammonium	mg/g				
Nitrate plus Nitrite	mg/g				
Total Phosphorus	mg/g				
Total Solids	%				
Arsenic	ug/g				
Cadmium ug/g					
Chromium	ug/g				
Cobalt	ug/g				
Copper	ug/g				
Lead ug/g					
Mercury	ug/g				
Molybdenum	ug/g				
Nickel	ug/g				
Potassium	ug/g				
Selenium	ug/g				
Zinc	ug/g				

B4.2 Sludge Digestion and Stabilization

[ ] Aerobic Digestion [ ] Anaerobic Digestion [ ] No Digestion

B4.3 Is sludge utilized on agricultural lands?

[ ] Yes [ ] No

Comments Area - B4 Sludge Quality

B5 ACUTE TOXICITY TEST

B5.1 Report the final effluent acute lethality monitoring test results in the following table:

Month	Sampling Date (dd/mm/yy)	Disinfection Process (circle one)	Acute Lethality Test Result	
			Rainbow Trout	Daphnia Magna
January	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
February	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
March	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
April	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
May	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
June	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
July	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
August	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
September	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
October	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
November	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail
December	/ /	CH/CD/UV/NO/OT	Pass / Fail	Pass / Fail

Disinfection Process Code:

CH = Chlorination
CD = Chlorination plus Dechlorination
UV = Ultra Violet Disinfection
NO = No Disinfection
OT = Others, please specify ______________________

B5.2 What is the sampling frequency for acute toxicity test required for your facility?

[ ] Monthly
[ ] Quarterly
[ ] Semi-annually
[ ] Annually

Comments Area - B6 Acute Toxicity Test Result

SECTION C - SELF ASSESSMENT

C 1 EFFLUENT LIMITS / PERFORMANCE

C1.1 Effluent Limits

C1.1.1 If your facility has a Certificate of Approval (C of A), please provide the effluent limits requirements as specified on the C of A:

Parameter	Monthly Average		Annual Average		Per Discharge Period Average		Others, please specify ( )		
	Concen. mg/L	Loading kg/d *	Concen. mg/L	Loading kg/d *	Concen. mg/L	Loading kg/d *	Conc. mg/L	Conc. % R	Loading kg/d *
BOD_5									
SS									
TP									
NH_3-N									
TKN									
pH		NA		NA		NA			NA
R. Cl_2		NA		NA		NA			NA
E. Coli		NA		NA		NA			NA

* specify the loading unit if it is not in kg/d, such as kg/year, kg/half year, etc - ( ). NA - Not Applicable. ***Attach a copy***

C1.1.2 If your facility does not have a Certificate of Approval (C of A) effluent limit requirement, the MOE Procedure F-5-1 requirements are used to assess the performance of your facility. Fill in the effluent requirements.

Type of Treatment		BOD_5 Annual Average	TSS Annual Average	TP Monthly Average
PRIMARY				
without	TP removal	30% removal	50% removal	Not Required
with	TP removal	50% removal	70% removal	1.0 mg/L
SECONDARY & TERTIARY				
without	TP removal	25 mg/L	25 mg/L	Not Required
with	TP removal	25 mg/L	25 mg/L	1.0 mg/L
LAGOONS				
without	TP removal	30 mg/L	40 mg/L	Not Required
continuous	TP removal	30 mg/L	40 mg/L	1.0 mg/L
batch	TP removal	25 mg/L	25 mg/L	1.0 mg/L
Your Plant's Effluent Requirements		⊙ mg/L or % removal	⊙ mg/L or % removal	⊙ mg/L

C1.2 C OF A LIMITS ASSESSMENT (If your facility has a C of A with effluent limits)

C1.2.1 Compare the effluent concentration and loading figures as provided in Section B2.1, B2.3 and/or B2.4 to the effluent limits in C1.1.1. Circle the compliance code in the following table that best describe the compliance status. The Overall Compliance is “Y” if the compliance status for all required parameters are either “Y”, “NR” or “ND”. The Overall Compliance is “N” if any one of the compliance status is “N” or “ID”.

Parameter	Effluent Compliance Status		
	Concentration	Loading	Both
$CBOD_5$	Y / N / ID / NR / ND	Y / N / ID / NR / ND	Y / N / ID / NR / ND
TSS	Y / N / ID / NR / ND	Y / N / ID / NR / ND	Y / N / ID / NR / ND
TP	Y / N / ID / NR / ND	Y / N / ID / NR / ND	Y / N / ID / NR / ND
TAN	Y / N / ID / NR / ND	Y / N / ID / NR / ND	Y / N / ID / NR / ND
TKN	Y / N / ID / NR / ND	Y / N / ID / NR / ND	Y / N / ID / NR / ND
pH	Y / N / ID / NR / ND	NA	Y / N / ID / NR / ND
R. Cl_2	Y / N / ID / NR / ND	NA	Y / N / ID / NR / ND
E. Coli	Y / N / ID / NR / ND	NA	Y / N / ID / NR / ND
OVERALL COMPLIANCE	NA	NA	Y / N / ID / NR / ND

Compliance Codes: Y = Yes, comply; N = No, not comply; ID = Insufficient Data; NA = Not Applicable; ND = No Discharge (or no direct discharge); NR = No Requirement

OVERALL COMPLIANCE

[ ] Y or NR or ND.. 0 points
[ ] N or ID .... 60 points

C1.3 MOE POLICY GUIDELINES ASSESSMENT (If your facility does not have a C of A with effluent limits) *(SECTION SUBJECT TO REVISION)*

C1.3.1 Compare the effluent concentration figures as provided in Section B2.1 to the effluent limits in C1.1.2 Circle the compliance code in the following table that best describe the compliance status. The Overall Compliance is “Y” if the compliance status for all required parameters are either “Y”, “NR” or “ND”. The Overall Compliance is “N” if any one of the compliance status is “N” or “ID”.

Parameter		Effluent Compliance Status Concentration
BOD_5	Annual Average	Y / N / ID / NR / ND
SS	Annual Average	Y / N / ID / NR / ND
TP	Monthly Average	Y / N / ID / NR / ND
OVERALL COMPLIANCE		Y / N / ID / NR / ND

Compliance Codes - refer to Section C1.2.1 above.

OVERALL COMPLIANCE

[ ] Y or NR or ND .... 0 points
[ ] N or ID .... 60 points

C1.4 PLANT PERFORMANCE EVALUATION

C1.4.1 Please provide Expected Effluent concentration requirements below. Refer to Section C1.1.1 for C of A requirements (use Monthly Average if available, otherwise use Annual Average or Per Discharge Period Average). If your facility does not have a C of A requirement, use Section C1.1.2 MOE Procedure F-5-1 requirements.

Parameter	Expected Effluent Concentration Requirements	
$CBOD_5$	____________________	mg/L
TSS	____________________	mg/L
TP	____________________	mg/L

C1.4.2 Compare the monthly average effluent concentration data provided in B2.1 to the above expected effluent concentration requirements. Count the number of months that the effluent concentration exceed the expected effluent requirements and complete the point score column below:

Exceedance and Point Scores	No. of Months Exceeding						Point Score
	0	1	2	3	4	5 & >	
Effluent concentration exceeds 90% effluent limits							
$CBOD_5$	0	5	5	10	10	15	
TSS	0	5	5	10	10	15	
TP	0	5	5	10	10	15	
Effluent concentration exceeds 100% effluent limits							
$CBOD_5$	0	5	5	10	10	15	
TSS	0	5	5	10	10	15	
TP	0	5	5	10	10	15	
Total Points							

C1.4.3 If your facility has a Seasonal Discharge, the number of exceeding months will be expressed as a percentage of the total number of discharging months. Use the following table instead of C1.4.2:

Exceedance and Point Scores	Percentage of Discharge Months Exceeding					Point Score
	0%	1 - 25%	26 - 50%	51-75%	> 75%	
Effluent concentration exceeds 90% effluent limits						
$CBOD_5$	0	5	5	10	15	
TSS	0	5	5	10	15	
TP	0	5	5	10	15	
Effluent concentration exceeds 100% effluent limits						
$CBOD_5$						
TSS	0	5	5	10	15	
TP	0	5	5	10	15	
	0	5	5	10	15	
Total Points						

TOTAL POINT VALUE FOR SECTION C1:

Section	C1.2.1 OR C1.3.1		C1.4.2 OR C1.4.3			Total
Point Score						

Comments Area - C1 Effluent Limits / Performance

C 2 PLANT CAPACITY

C2.1 Capacity - Existing

C2.1.1 List below the design Average Daily Flow (ADF) and the average BOD_5 / SS loadings for your facility.

Design Average Daily Flow = ________________ 1,000 m^3/d (90% = ________________)
Design BOD_5 Loading = ________________ kg/d (90% = ________________)
Design TSS Loading = ________________ kg/d (90% = ________________)

If the design TSS loading is unknown, use a factor of 1.2 times the design BOD_5 loading.

C2.1.2 Count the number of months the ADF, or BOD_5 loading, or TSS loading (as reported in Section B2.3) that exceed the design figure in C2.1.1. For Primary STPs, count ONLY the number of months the ADF exceeds the design flow.

Exceedance and Point Scores	No. of Months Exceeding						Point Score
	0	1	2	3	4	5 & >	
ADF exceeds 90% of design flow	0	0	0	5	5	10	
ADF exceeds 100% of design flow	0	5	5	10	10	20	
BOD_5 loading exceeds 90% of design load	0	0	0	5	5	10	
BOD_5 loading exceeds 100% of design load	0	5	5	10	10	20	
TSS loading exceeds 90% of design load	0	0	0	5	5	10	
TSS loading exceeds 100% of design load	0	5	5	10	10	20	
Design ADF, or BOD_5 Loading, or TSS Loading UNKNOWN	90						
Total Points							

C2.2 Capacity - Projected 5 Year

C2.2.1 Average Daily Flow and Loading Trends

Year	Average Daily Flow (1,000 m^3/d)	Influent BOD_5		Influent TSS		Design Capacity		
		Concen. (mg/L)	Loading (kg/d)	Concen. (mg/L)	Loading (kg/d)	ADF (1,000 m^3/d)	BOD_5 Loading (kg/d)	TSS Loading (kg/d)
Previous Year								
Current Report Year								
5 Year Projection *						*	*	*
% of Design Capacity	%				%			

* Projection is based on the Official Plan. Design Capacity includes planned expansion / upgrade for your facility.

C2.2.2 What is the "% of Design Capacity" for the 5 Year Projected ADF, or BOD_5 loading, or TSS loading (use the greater of the three %) in Section C2.2.1 above?

- [ ] Less than 90% .... 0 points
- [ ] 90% - 100% .... 10 points
- [ ] 101% - 110% .... 30 points
- [ ] 111% - 120% .... 60 points
- [ ] Greater than 120% OR UNKNOWN .... 90 points

TOTAL POINT VALUE FOR SECTION C2:

Section	C2.1.2	C2.2.2				Total
Point Score						

Comments Area - C2 Plant Capacity

C 3 CSOs AND PLANT BYPASSES

C3.1 CSOs

C3.1.1 How many times in the last calendar year bypasses or overflows occurred in any part of the sewerage system (refer to B3.1 page 9)?

- [ ] 2 times or less........ 0 points
- [ ] 3 times........ 5 points
- [ ] 4 times........ 10 points
- [ ] 5 or more times........ 20 points

C3.2 Plant Bypasses

C3.2.1 What is the total volume of bypasses (Primary + Secondary) expressed as percentage of the Average Daily Flow (refer to B3.3 page 11)?

- [ ] 0%, no bypassing........ 0 points
- [ ] Greater than 0% to less than 1%........ 5 points
- [ ] 1% to less than 5%........ 10 points
- [ ] 5% to less than 10%........ 20 points
- [ ] 10% and above, OR bypass volume unknown........ 40 points

C3.3 Sewage Generation

C3.3.1 Estimate the sewage flow generated by Industrial, Commercial and Institutional establishments by areas:

Type of Establishment	Area ha	Design Flow Figure $m^3/ha.d$	Sewage Flow Generated * 1,000 m^3/d
Commercial / Institutional		28	
Light Industrial		35	
Heavy Industrial		55	
Total Industrial / Comm. / Institution			

* Sewage Flow Generated (1,000 m^3/d) = Area (ha) X Design Flow Figure ($m^3/ha.d$) ÷ 1,000

C3.3.2 Calculate the Domestic Sewage Generation Rate based on the annual average daily flow minus the industrial / commercial / institutional flows and then divided by the population served by your facility.

Domestic Sewage Generation, litre/capita/day

= (Average Daily Flow - Industrial / Commercial / Institutional Flows) in 1,000 m^3/d ÷ Population Served X 1,000,000

= (-   ) ÷ ____________ X 1,000,000

= __________ litre/capita/day

C3.3.3 What is the calculated Domestic Sewage Generation Rate for your facility?

[ ] Less than 450 L/c/d (100 igpcd) .... 0 points
[ ] 450 L/c/d to less than 680 L/c/d (150 igpcd)........ 5 points
[ ] 680 L/c/d to less than 910 L/c/d (200 igpcd)........ 10 points
[ ] 910 L/c/d (200 igpcd) and above .... 15 points

C3.4 Sewage Flow Peaking Factor

C3.4.1 Calculate the Peaking Factor based on the maximum instantaneous sewage flow rate (registered by the flow meter at the head work before bypassing) divided by the annual average daily flow.

Peaking Factor = Maximum Flow Rate ÷ Annual Average Daily Flow

= ÷

= ______________

C3.4.2 What is the calculated Peaking Factor for your facility?

[ ] Less than 4 .... 0 points
[ ] 4 to less than 5 .... 5 points
[ ] 5 to less than 6 .... 10 points
[ ] 6 and above........ 15 points

TOTAL POINT VALUE FOR SECTION C3:

Section	C3.1.1	C3.2.1	C3.3.3	C3.4.2		Total
Point Score						

Comments Area - C3 CSOs and Plant Bypasses

C 4 SLUDGE HANDLING

This Part is not applicable to LAGOON systems where sludge is not removed on a regular basis.

C4.1 Sludge Final Disposal

C4.1.1 Method of sludge disposal:

[ ] Incineration - GO DIRECTLY TO SECTION C4.4
[ ] Landfill
[ ] Utilization on Agricultural Land
[ ] Others, specify ______________

C4.2 Sludge Stabilization

C4.2.1 Method of sludge stabilization used:

[ ] Aerobic digestion .... 0 points
[ ] Anaerobic digestion - single stage .... 0 points
[ ] Anaerobic digestion - multi stages .... 0 points
[ ] Others, specify ______________ .... 0 points
[ ] No stabilization .... 10 points

C4.3 Sludge Storage

C4.3.1 How many months of sludge storage capacity are available for your facility, either on-site or off-site? Off-site capacity includes approved sludge disposal at landfills where sludge stabilization is not a prerequisite.

[ ] 6 months or more .... 0 points
[ ] 4 months to less than 6 months .... 5 points
[ ] 3 months to less than 4 months .... 10 points
[ ] 2 months to less than 3 months .... 15 points
[ ] Less than 2 months .... 20 points

C4.4 Sludge Disposal Approval

C4.4.1 For how much longer does your facility have access to and approval of final sludge disposal?

[ ] 3 years or more .... 0 points
[ ] 2 years to less than 3 years .... 5 points
[ ] 1 year to less than 2 years .... 10 points
[ ] 1/2 year to less than 1 year .... 15 points
[ ] Less than 1/2 year .... 20 points

C4.5 Sludge Accountability

C4.5.1 Sludge accountability analysis is a comparison of expected and actual sludge productions. Please provide the required information for the sludge accountability analysis below:

Anticipated Sludge Production

Sludge Received from Other Facilities

Sludge Discharge Point [ ] Plant Inlet [ ] Other location ____________________

TO BE COMPLETED ONLY IF SLUDGE DISCHARGE POINT = Plant Inlet, ELSE (A) = 0 :

Average Sludge Concen.	Sr	=	mg/L
Sludge Received		= Vr X Sr	
		= ____________	kg/year.. (A)

Primary Sludge Production

Average Daily Flow	Q	=	1,000 m^3/d
Average Influent SS	SSi	=	mg/L
% TSS Removal	R	=	% (actual % removal or from Table 1 Page 24)
Primary Sludge Produced		= Q X SSi X R / 100 X 365	
		= ____________	kg/year.. (B)

Biological Sludge Production

Primary Effluent BOD_5	BODs	=	mg/L
Secondary Effluent BOD_5	BODe	=	mg/L
Sludge Production Value	Ks	=	kg TSS / kg BOD_5 removed (from Table 2 Page 24)
Biological Sludge Produced		= Q X Ks X (BODs - BODe) X 365	
		= ____________	kg/year.. (C)

Chemical Sludge Production

Chemical Used	[ ] Dry Alum	[ ] Liquid Ferric Chloride
	[ ] Liquid Alum	[ ] Liquid Ferrous Chloride
	[ ] Dry Sodium Aluminate	[ ] Dry Ferrous Sulphate
	[ ] Liquid Sodium Aluminate	[ ] Others, ____________

Chemical Dosage, <u>if Liquid Chemical</u> = Chemical feed rate X Density X Metal % (actual or from Table 3 Pg 25)

Primary	=	m^3/d	X	kg/m^3	X	÷ 100 X 365 days
Secondary	=	m^3/d	X	kg/m^3	X	÷ 100 X 365 days
Total Cd	=			kg/year		

<u>if Solid Chemical</u> = Chemical feed rate X Metal % (actual or from Table 3 Page 25)

Primary	=	kg/d	X	÷ 100 X 365 days
Secondary	=	kg/d	X	÷ 100 X 365 days
Total Cd	=			kg/year

Sludge Production Ratio Cr = 4.79 kg TSS / kg Al added or 2.87 kg TSS / kg Fe added

Chemical Sludge Produced = Cr X Cd

= ____________ kg/year.. (D)

Total Anticipated Sludge Production = (A) + (B) + (C) + (D)
= ____________ kg/year .... (1)

Actual Sludge Removal

Primary Sludge

Volume of Sludge Removed Vp = 1,000 m^3/year
Sludge Concentration Sp = mg/L (actual concentration or from Table 4 Page 25)
Primary Sludge Removed = Vp X Sp
= ____________ kg/year .... (E)

Secondary Sludge

Volume of Sludge Wasted * Vs = 1,000 m^3/year (* Not to Primary Clarifier)
Sludge Concentration Ss = mg/L (actual concentration or from Table 4 Page 25)
Secondary Sludge Wasted = Vs X Ss
= ____________ kg/year .... (F)

Effluent Suspended Solids

Average Daily Flow Q = 1,000 m^3/d
Effluent TSS Concentration SSe = mg/L
Effluent TSS = Q X SSe X 365
= ____________ kg/year .... (G)

Bypass Suspended Solids

Total Volume of Bypass Vb = 1,000 m^3/year
Average TSS Concentration SSb = mg/L
Bypass TSS = Vb X SSb
= ____________ kg/year .... (H)

Total Sludge Accounted-for = (E) + (F) + (G) + (H)
= ____________ kg/year .... (2)

Sludge Unaccounted-for = (1) - (2)
= ____________ kg/year .... (3)

Sludge Unaccounted-for Percent = (3) / (1) X 100%
= ____________ % .... (4)

Interpretation:

The Point Score for this Part is equal to the percentage of Sludge Unaccounted-for as calculated in line 4 above. Each one percent of Sludge Unaccounted-for equals to 1 point, round-up to the nearest 1 and subject to a maximum of 50 points.

Sludge Accountability Points .... points

TOTAL POINT VALUE FOR SECTION C4:

Section	C4.2.1	C4.3.1	C4.4.1	C4.5.1		Total
Point Score						

Comments Area - C4 Sludge Handling

Tables

Table 1: Percentage Removal of TSS by Primary Clarifier Based on Surface Overflow Rate (SOR)..... R Value

Surface Loading Rate (SOR)		% TSS Removal R Value
gal/day/sq ft.	cu.m/day/sq m.	
0 - 1,000	0 - 40	65%
1,000 - 1,500	40 - 60	45%
1,500 - 2,000	60 -80	30%
> 2,000	> 80	no removal

Table 2: Standard Biological Sludge Production Value ..... Ks Value

FOR SUSPENDED GROWTH PROCESSES		FOR FIXED FILM PROCESSES	
Process Type	kg TSS / kg BOD_5 Removed	Process Type	kg TSS / kg BOD_5 Removed
Activated Sludge with primary clarification	0.70	Trickling Filter	0.90
Activated Sludge without primary clarification		Rotating Biological Contactor (RBC)	1.00
Conventional, includes tapered aeration, step feed, plug flow and complete mix with detention time < 10 hrs.	0.85	Activated Bio-Filter (ABF)	1.00
Extended Aeration, includes oxidation ditch	0.65		
Contact Stabilization	1.00		

Table 3: Density and Metal % of Aluminum and Iron Salts

The following table lists the typical values of the density and metal contents for some commonly used aluminum and iron salts. You should try to obtain these values from your chemical manufacturer. In the absence of these values, you may use the average values as provided on the following table to estimate the chemical sludge production.

CHARACTERISTICS OF SOME COMMONLY USED ALUMINUM AND IRON SALTS				
Common Name	Formula	Density, kg/m^3 (average value)	Commercial Strength % by weight	Metal % by weight (average value)
Dry Alum	$Al_2(SO_4)_3.14H_2O$	600 - 1200	17% Al_2O_3	9.0% Al
Liquid Alum	$Al_2(SO_4)_3.14H_2O$	1330 @ 16°C	8.3% Al_2O_3	4.4% Al
Dry Sodium Aluminate	$Na_2Al_2O_4$	640 - 800	41-46% Al_2O_3	21.7-24.4% Al (23.1)
Liquid Sodium Aluminate	$Na_2Al_2O_4$	-	4.9-26.7% Al_2O_3	2.6-14.2% Al (8.4)
Liquid Ferric Chloride	$FeCl_3$	1340 - 1490 (1415)	35-45% $FeCl_3$	12.0-15.5% Fe (13.8)
Liquid Ferrous Chloride	$FeCl_2$	1190 - 1250 (1220)	20-25% $FeCl_2$	8.8-11.0% Fe (9.9)
Dry Ferrous Sulphate	$FeSO_4.7H_2O$	990 - 1060	55-58% $FeSO_4$	20.2-21.3% Fe (20.8)

Table 4: Sludge Concentrations for Projecting Sludge Wastage from Primary and Secondary Clarifier ..... Sp and Ss Values

Sludge Type	Sludge Concentration Sp or Ss Values (mg/L)
Primary	50,000
Secondary	
Return Activated Sludge / Conventional	6,000
Return Activated Sludge / Extended Aeration	7,500
Return Activated Sludge / Contact Stabilization	8,000
Return Activated Sludge / Small plant with low SOR	10,000
Separate waste hopper in secondary clarifier	12,000

C 5 EFFLUENT SAMPLING / ANALYSIS

C5.1 Sampling

C5.1.1 How many effluent samples per month (per discharge period in the case of seasonal discharge lagoons) were submitted for analysis for compliance purpose?

[ ] More than 1 sample per month .... 0 points
[ ] 1 sample per month .... 5 points
[ ] Less than 1 sample per month .... 10 points

C5.1.2 How do you select the date of Effluent Compliance Sampling?

[ ] Follow strictly to a fixed sampling schedule .... 0 points
[ ] Samples are collected at random without a pre-determined schedule........ 10 points

C5.1.3 How were the samples collected?

If your facility is a continuous discharge plant with a design capacity of 1 MIGD (4,500 m^3/d) or above:

[ ] 24-hour flow proportional or equal volume composite .... 0 points
[ ] Less than 24-hour composite .... 5 points
[ ] Single grab sample .... 10 points

If your facility is a continuous discharge plant with a design capacity of less than 1 MIGD (4,500 m^3/d):

[ ] 8-hour (or more) flow proportional or equal volume composite .... 0 points
[ ] Less than 8-hour composite .... 5 points
[ ] Single grab sample .... 10 points

If your facility has a seasonal discharge:

[ ] 3 grab samples (or more) taken one on the first day, one in the middle and one on the final day of discharge, per discharge period .... 0 points
[ ] 2 samples taken per discharge period .... 5 points
[ ] Less than 2 samples taken per discharge period .... 10 points

C5.2 Laboratory Analysis

C5.2.1 How long after collection are the samples shipped to the Laboratory?

[ ] Within 24 hours .... 0 points
[ ] More than 24 hours .... 5 points

C5.2.2 Are samples kept at a temperature above the freezing point of the samples and under 10 degree C during the sampling period and transportation to the Laboratory?

[ ] Yes .... 0 points
[ ] No .... 5 points

C5.2.3 Does the Laboratory that analyzed your effluent samples have a Quality Assurance / Quality Control (QA/QC) program in place?

[ ] Yes, QA/QC program is in place .... 0 points
[ ] No or Don't know whether QA/QC program is in place .... 10 points

TOTAL POINT VALUE FOR SECTION C5:

Section	C5.1.1	C5.1.2	C5.1.3	C5.2.1	C5.2.2	C5.2.3	Total
Point Score							

Comments Area - C5 Effluent Sampling / Analysis

C 6 EQUIPMENT MAINTENANCE

C6.1 Preventive Maintenance

C6.1.1 Does your facility adhere to a written preventive maintenance schedule on major equipment?

[ ] Yes .... 0 points
[ ] No .... 20 points

C6.1.2 Does this preventive maintenance schedule list the frequency / interval for the routine maintenance work and the materials (e.g. type of lubricant, filter, parts, etc.) required for the task?

[ ] Yes .... 0 points
[ ] No .... 5 points

C6.1.3 Are these preventive tasks, as well as other repair works / equipment problems being recorded and filed for future reference?

[ ] Yes .... 0 points
[ ] No .... 5 points

C6.2 Sewage Flow Meter Calibration

C6.2.1 Have the influent and/or effluent flow meter(s) been calibrated in the last year? Primary element refers to flumes, weirs, Venturi, Magnetic meters, etc. Secondary element refers to flow converters, bubblers, transducers, recorders, indicators, totalizers, data loggers, etc.

[ ] Yes - Both Primary and Secondary Elements, Calibrated on ______________ .... 0 points
[ ] Yes - Secondary Element ONLY, Calibrated on ______________ .... 5 points
[ ] Yes - Primary Element ONLY, Calibrated on ______________ .... 10 points
[ ] No OR flow meter not installed .... 20 points

TOTAL POINT VALUE FOR SECTION C6:

Section	C6.1.1	C6.1.2	C6.1.3	C6.2.1		Total
Point Score						

Comments Area - C6 Equipment Maintenance

C 7 OPERATOR TRAINING AND CERTIFICATION

C7.1 What is the Classification of your facility under Regulation 129/04?

[ ] Class I [ ] Class II [ ] Class III [ ] Class IV

C7.2 What is the name, classification and certification number for the operator-in-charge (OIC) with overall responsibility for your facility?

Name of OIC: ______________________

OIC Operator's Licence Class: [ ] Class I [ ] Class II [ ] Class III [ ] Class IV

Certification Number:______________________

C7.3 Was the OIC with overall responsibility certified at the same classification or higher as the facility?

[ ] Yes .... 0 points
[ ] No .... 20 points

C7.4 Were all other operators in the facility certified under Regulation 129/04?

[ ] Yes .... 0 points
[ ] No .... 20 points

C7.5 How many hours of training has been provided to each operator during the last calendar year as required under Section 21 of Regulation 129/04? Training can be courses, conferences, or in-house training.

[ ] 40 hours or more .... 0 points
[ ] Less than 40 hours .... 10 points

TOTAL POINT VALUE FOR SECTION C7:

Section	C7.3	C7.4	C7.5			Total
Point Score						

Comments Area - C7 Operator Training / Certification

C 8 FINANCIAL STATUS

C8.1 Operation and Maintenance Budget

C8.1.1 Please provide the Operation and Maintenance budget for your facility:

Budget / Expenses	Last Year Budget	Last Year Actual Expenditure	Present Year Budget
Total Operation and Maintenance Budget / Expenditure	$	$	$

C8.1.2 Is the budget for the present year sufficient to support normal operation and maintenance of the facility?

[ ] Yes.. 0 points
[ ] No .... 20 points

C8.1.3 Are contingency funds available for unforeseen expenses such as major equipment replacement, etc.?

[ ] Yes.. 0 points
[ ] No .... 10 points

C8.2 Reserved Fund

C8.2.1 Is there a reserved fund system to provide funding for facility improvement / upgrade / expansion needs?

[ ] Yes.. 0 points
[ ] No .... 20 points

TOTAL POINT VALUE FOR SECTION C8:

Section	C8.1.2	C8.1.3	C8.2.1			Total
Point Score						

Comments Area - C8 Financial Status

APPENDIX 1

SELF ASSESSMENT REPORT

GUIDE

PAGE 2 - REPORT SUMMARY

Fill in the “Actual Value” column. Enter the total on the Total Point Score box.

Compare the “Actual Value” to the “Action Level” and the “Total Score” to the Report Point Range. Put a check mark (_) against the one that matches your score.

If any of the “Actual Value” equals to or exceeds the value in the “Action Level” column, put the check mark against the 200 - 720 points range irrespective of the total point score.

PAGE 3 - COMMENTS BY OPERATING AUTHORITY / OWNER / MUNICIPALITY

If the "ACTUAL VALUE" scored by your facility equals or exceeds the value in the "ACTION LEVEL", you may provide an explanation for this situation such as cause, actions taken / proposed, etc.

PAGE 4 - PLANT INFORMATION

This page would be pre-printed with information for your facility as submitted last year (if this is not your first submission). Please verify the information and make corrections if necessary.

Works Number refers to the unique identification number for your facility given by MOE.

Certificate of Approval (C of A) refers to the approval document issued by the Ministry of the Environment to signify the approval for the construction and operation of your facility. Not every plant has a C of A. If your facility does not have a C of A, put "NA" under the Certificate of Approval No.

Facility Classification refers to the classification of your facility under Regulation 129/04 Licensing of Sewage Works Operators. Classification is from I to IV where IV is being the most complex.

Ministry Region refers to the five Ministry regions, namely Southwestern, West Central, Central, Eastern, and Northern, where the plant is geographically located.

District refers to the name of your Region, County or District which your facility serves.

Municipality refers to the name of your Municipality which your facility serves.

Treatment Process Description - please check the treatment description as provided and re-select the Major Process, Additional Treatment, Discharge Mode, Disinfection Process and Phosphorus Removal descriptions and codes from the following table. These codes and descriptions are currently used in the UMIS (Utility Monitoring Information System), with some additions.

Code	***Major Process***	***Code***	***Additional Treatment***
001	Primary	015	Effluent Polishing
002	Conventional Activated Sludge	018	Effluent Filtration
003	Modified Activated Sludge	019	Polishing Lagoon(s)
004	Contact Stabilization	020	Odour Control
005	Extended Aeration	021	Polishing Clarifier
006	Trickling Filter	022	Adsorption
007	High Rate	093	Denitrification
008	Extended Aeration / Contact Stabilization	103	Rotating Biological Contactor (effluent polishing)
009	Convertible Operating Mode	104	Nitrification
010	Oxidation Ditch	135	Polishing Lagoon(s) Seasonal
011	Aerated Cell	136	Polishing Lagoon(s) Continuous
012	Communal Septic Tank		
013	Individual Septic Tank	***Code***	***Discharge Mode***

075	Aerated Lagoon	023	Regulated Discharge Volume
076	Anaerobic Lagoon	082	Annual Discharge
077	Conventional Lagoon Continuous	083	Seasonal Discharge
078	Conventional Lagoon Seasonal	084	Continuous Discharge
079	Lagoon and Spray	089	No Discharge
080	Aerated Cell Plus Lagoon	090	Complete Retention
081	Conventional Lagoon Annual	094	Exfiltration
085	Exfiltration Lagoon	100	Spray Irrigation
133	Rotating Biological Contactor	137	Summer Storage
138	Sutton Process		
139	New Hamburg Process		
140	Sequencing Batch Reactor	***Code***	***Effluent Disinfection Practice***
		017	Chlorination + Dechlorination, Seasonal
		217	Chlorination + Dechlorination, Year Round
Code	***Phosphorus Removal***	048	Chlorination, Seasonal
016	Phosphorus Removal - Continuous	248	Chlorination, Year Round
106	Phosphorus Removal - Batch	200	No Disinfection
		201	UV Disinfection, Seasonal
		202	UV Disinfection, Year Round
		210	Other Disinfection Process

Design Capacity is the average daily flow that your facility is designed for (in 1,000 m^3/day).

Design Population is the number of people that your facility is designed for.

Population Served is the actual number of people that your facility is serving. This population figure is normally available from your Planning Department, through census or by estimation based on the number of houses in each sub-division in your municipality.

Watercourse is the name of the receiving body of water that your facility is directly discharged into.

Major Basin refers to Great Lakes, Arctic Watershed and Nelson River drainage basins.

Minor Basin refers to the minor basin that the receiving watercourse connected to. The eight minor basins include Lake Superior, Lake Huron, Lake Erie, Lake Ontario, St. Lawrence River, Ottawa River, James Bay and Lake Winnipeg East.

UTM Coordinates refer to the geo-reference of the location of your facility. The coordinates of your facility can be read directly from a topographic map with UTM marking on the sides.

<u>PAGE 6 - B1 INFLUENT FLOW / CONC / LOADING</u>

List the maximum, minimum and average daily flow in the month and the average concentration of the raw sewage received at your facility. All flows are in 1,000 m^3/day. If ammonia concentration data is not available, leave the column blank.

The monthly average influent loading, in kg/day is equal to the monthly average daily flow, in 1,000 m^3/day times the monthly average influent concentration of the parameter, in mg/L.

PAGE 7 - B2 EFFLUENT CONC / LOADING

Provide the monthly average effluent concentration for the parameters that were measured. Leave the other columns blank if the parameter is not tested.

If your facility has a seasonal discharge and the discharge period overlaps two calendar months, you can report the effluent quality based on an average over the discharge period, instead of monthly, and report the data on the month when the discharge begins.

Discharge Duration is the total number of hours of discharge. The Volume of Discharge is the total volume of discharge during that discharge period, in 1,000 m^3.

The ***effluent loading*** is calculated by multiplying the monthly average daily flow with the corresponding monthly average effluent concentration.

Effluent Load per discharge period is calculated by multiplying the volume of discharge with the average effluent concentration during that discharge period. The loading discharge rate is obtained by dividing the effluent load per discharge period by the number of days with discharges. The ***Maximum Discharge Rate*** is the maximum figure from the column.

PAGE 9 - B3 CSOs AND PLANT BYPASSES

If your plant operation staff is not responsible for the sewage collection system, you may consult the appropriate authority in your municipality on this section of the Report.

CSO is the discharge of raw sewage directly into surface water from a combined sewer collection system. The construction of combined sewer is normally not approved after 1956. However, the combined sewers built before this time are still in operation in many municipalities in Ontario. Information collected in this section would indicate whether CSOs have posed any significant problems to your municipality.

Plant Bypass means the discharge of raw sewage that occurred within the boundary of your treatment facility. Bypasses at the last pumping station of lagoons are considered to be plant bypasses. For the purpose of this Report, bypasses are classified into primary and secondary. *Primary bypass* means the discharge of raw sewage subject to no treatment except grit removal and/or disinfection. *Secondary bypass* means the discharge of sewage that has undergone solids removal at the primary clarifier but bypassed the secondary treatment process. As some of the older plants do not have a bypass flow meter installed, estimates on bypass duration and volume are acceptable as long as they are based on some documented procedures.

PAGE 12 - B4 SLUDGE QUALITY

If sludge (biosolids) is utilized on agricultural lands, sample quarterly. Otherwise, sample yearly.

If more than one sample is tested, report the average of the samples for the quarter or for the year, depending on the sampling frequency that is required.

If sludge is anaerobically digested, report all parameter test results. Otherwise, report all parameters with the exception of the first three, i.e. pH, Ammonia plus Ammonium, Nitrate plus Nitrite.

Please note that all metal results are in micro-gram per gram, ug/g.

PAGE 13 - B5 ACUTE TOXICITY TEST RESULT

Report the final effluent acute toxicity test result, the date of sampling and the disinfection process in use at the time of sampling.

PAGE 14 - C1 EFFLUENT LIMITS / PERFORMANCE

If your facility has a Certificate of Approval with non-compliance limits, provide these limits in C1.1.1. If the limits are not based on averages of monthly, annually or per discharge period, please specify the average period on the last column and attached a copy of the C of A.

If your facility does not have a C of A with non-compliance limits, complete C1.1.2 instead of C1.1.1. This table provides the expected effluent quality of your facility.

C of A Limits Assessment -This is strictly a numeric comparison of the effluent quality against the C of A limits. Any exceedance or insufficient data constitutes non-compliance.

MOE Procedure F-5-1 Assessment - This is a numeric comparison of the effluent quality against Procedure F-5-1. Any exceedance or insufficient data means that your facility has failed to meet the requirements.

Plant Performance Evaluation - The Expected

Effluent concentration is the most stringent requirement that your facility is expected to meet. It is used to evaluate the significance of non-compliance if your effluent quality is near or exceed the expected effluent requirements. The point scores for 90% and 100% limits exceedance are additive. Point scored at 90% limits exceedance serves as an early warning for potential non-compliance occurrence.

PAGE 17 - C2 PLANT CAPACITY

The ***Design Average Daily Flow*** is the hydraulic capacity that the plant is designed for. Use the approved "rated capacity" on your Certificate of Approval as the design capacity for your facility. In the absence of a C of A, obtain these figures from your plant's design manual. The Design BOD_5 / TSS loadings are the organic loading capacity that the plant is designed to handle. In the absence of a TSS loading design criteria, use a factor of 1.2 times the design BOD_5 loading as the TSS loading capacity.

Your ***5 year projection*** on Average Daily Flow, BOD_5 and TSS loadings should be based on your Official Plan, which is available from your Planning Department. Compare these 5 year projection figures to the design capacity of your facility.

PAGE 19 - C3 CSOs AND PLANT BYPASSES

Sewage generation rate is a calculation of the average volume of sewage generated by each person every day. The calculated sewage generation rate includes a proportion of flow contributed by inflow and infiltration (I/I) in the collection system. A high sewage generation rate would indicate that you may have a leaky sewage collection system.

C3.3.1 is used to estimate (in the absence of real flow data) the proportion of flow from industrial and commercial activities which is subtracted from the total flow to obtain the domestic flow.

Peaking factor is an indication of extraneous flow as a result of rain storm or snow melt event. A more leaky sewer collection system would have a higher peaking factor. The maximum flow rate used to calculate the peaking factor is the instantaneous maximum flow rate (NOT the maximum average daily flow) received at the headworks of your facility before any bypassing, during the reporting year.

PAGE 21 - C4 SLUDGE HANDLING

This section is not applicable to Lagoon systems. Skip this section If your facility is a Lagoon.

If your facility uses incineration for sludge disposal, skip C4.2 and C4.3.

Sludge Storage - Wasting of surplus activate sludge from the aeration system to maintain the optimal mass balance is a key process control parameter in the activated sludge treatment process. Facilities that do not have an adequate sludge storage capacity would face sludge wasting problem once the storage capacity is used up. Failure to control sludge mass balance would result in significant reduction in effluent quality. Thus, inadequate sludge storage capacity would become a major factor that limits the performance of your facility.

Sludge Disposal Approval - Your facility should always maintain a valid approval of and access to a final sludge disposal site for a reasonable period. Shortage of sludge disposal sites will create problems for your facility. Long term planning is needed to find, select and obtain approval for new sludge disposal site.

Sludge Accountability - Sludge Accountability is an assessment of reported plant performance by evaluating the sludge produced by your facility. The basic theory of this calculation is based on sludge mass balance in a biological process. Under steady state condition, "what is coming in" plus "what is produced" will be equal to "what is going out".

Anticipated Sludge Production ("what is coming in" plus "what is produced")

Sludge Received from Other Facilities - Some facilities receive sludge from other plants / sources. If the sludge discharge point is at the Plant Inlet, this amount of sludge is added to the biological process. Otherwise, the sludge is not contributing to the biological process and can be ignored.

Primary Sludge Production - This is the amount of sludge settled in the primary clarifier from raw sewage based on % TSS removal. If your facility analyzes primary tank Influent and effluent samples, you can use the actual annual average % TSS removal figure. Otherwise, use the R value from Table 1 on page 24 that corresponds to the surface loading rate of your primary clarifier. Surface loading rate ($m^3/day/m^2$) is the annual average daily flow (m^3/day) divided by the surface area of the primary clarifier (m^2).

Biological Sludge Production - The biological sludge produced is calculated based on the quantity of BOD_5 removed through the secondary biological process. Primary tank effluent BOD_5 is used to represent the secondary Influent BOD_5. The standard sludge production value, Ks is obtained from Table 2 page 24, that corresponds to the type of process used.

Chemical Sludge Production - If inorganic chemical, such as Alum or Iron salt is used for coagulation or Phosphorus removal in the Primary or Secondary Clarifiers, report the quantity of chemical used in a year. The chemical used should be expressed as kg/year as Fe or as Al, depending on which chemical is used. Chemical sludge produced is proportional to the quantity of chemical used and the ratio is given in the formula on page 22. Table 3 on page 25 lists the % Al or % Fe in some chemicals commonly used in STPs.

Sludge Removal ("what is going out")

Primary Sludge - Report the total volume of sludge removed from the primary clarifier in a year. The volume of sludge removed can be estimated by:

Sludge Haulage - No. of loads per year times the volume of each load.

Sludge Drying Beds - No. of beds per year times the volume of the beds.

Sludge Dewatering - Volume of sludge fed into the sludge dewatering units.

If your facility wastes secondary sludge to the primary clarifier, the quantity already includes both primary and secondary sludge. Sludge concentration is the annual average sludge concentration if sludge samples are analyzed. Otherwise, select the Sp value that corresponds to the sludge type in Table 4 page 25, i.e. 50,000 mg/l.

Secondary Sludge - Used this section only if the secondary sludge is NOT wasted to Primary Clarifier. The volume of sludge wasted is the total volume wasted in a year. Sludge concentration is the annual average concentration or the corresponding Ss value in Table 4 page 25 in the absence of actual data.

Effluent / Bypass Suspended Solids - Normally these figures represent only a very small percentage of the total mass of solids in the system. Bypass Suspended Solids will be considered in the calculation ONLY if the bypass occurred after the primary clarifier.

The difference between the Sludge Production and the Sludge Removal figures is the ***sludge mass un-accounted-for***. This is the amount of sludge most likely lost through the effluent outlet weir un-intentionally, such as during the peak flow of the day or during some process upset periods.

PAGE 26 - C5 EFFLUENT SAMPLING / ANALYSIS

Report the number of samples per month analyzed for the purpose of compliance assessment. If your facility has a seasonal discharge, report the number of samples analyzed per discharge period, instead of monthly.

A ***fixed sampling schedule*** could be decided at the beginning of each year so that effluent samples can be collected in a more representative manner. For a seasonal discharge plant, a fixed sampling date may not always be possible. In that case, one sample taken on the first day, one in the middle and one on the final day of discharge, can be accepted as having a fixed schedule.

Flow proportional composite sampling is the most desirable, followed by equal volume / equal time composite sampling. A single grab sample is the least desirable.

A competent laboratory should have a ***Quality Assurance / Quality Control (QA/QC)*** program in place. Your effluent samples should be analyzed by a competent laboratory. The Laboratory Manager or the person in charge of the Laboratory will be able to tell you whether they have an adequate QA/QC program in place.

PAGE 28 - C6 EQUIPMENT MAINTENANCE

A ***preventive maintenance schedule*** normally lists all major equipment maintenance work recommended by the manufacturer. Other information such as frequency, type of lubricants used, date scheduled, date completed, etc. will be provided on the schedule. Preventive maintenance helps to reduce equipment down time and to prolong equipment life.

Flow is a key concern in the design, approval and operation of a STP. Flow meters have to be calibrated periodically to maintain the designed accuracy. Sewage flow meters normally consist of a primary element and a secondary element. The primary element (e.g. flumes, weirs, Venturi, Magnetic meters, etc.) is used to generate a correlation between flow rate and some measurable criteria such as water depth, water head, magnetic flex, etc. The secondary element (e.g. flow converters, bubblers, transducers, recorders, flow indicators, totalizers, data loggers, etc.) is used to convert, record or totalize this measurable criteria into some flow units such as flow rate or volume. Both primary and secondary elements require calibration.

PAGE 29 - C7 OPERATOR TRAINING AND CERTIFICATION

Regulation 129/04 requires that sewage works (unless as exempted by the Regulation) shall be operated by an operator who holds an appropriate license that is of the same class or higher than the class of the facility. There are four classifications i.e. from I to IV. Class IV is the highest in terms of capacity and complexity of the treatment facility.

Operator-in-charge (OIC) is the person who has the overall responsibility for your facility. Normally the OIC is the Plant Manager or the Superintendent. Facilities with a lower classification may have the Supervisor or the Chief Operator as the OIC.

PAGE 30 - C8 FINANCIAL STATUS

The budget can be either approved or proposed (if not yet approved). Last Year refers to the Reporting Year. Present Year refers to the current year.

APPENDIX C

Managers Guide to Sewage Treatment Plant Optimization

Prepared for

Ontario Ministry of the Environment and Energy
Environment Canada
The Municipal Engineers Association
Water Environment Association of Ontario

Prepared by

Wastewater Technology Centre
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and

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Edited by

Water and Wastewater Optimization Section
Ontario Ministry of Environment and Energy

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The Great Lakes 2000 Cleanup Fund

The Cleanup Fund, part of the Government of Canada's Great Lakes 2000 Program, supports the development and implementation of cleanup technologies to restore beneficial uses in Canada's 17 Areas of Concern. The Cleanup Fund has provided about $43 million in support of more than 230 projects in the areas of sediment cleanup, stormwater and combined sewer overflow management, municipal wastewater treatment, and habitat rehabilitation. For more information or copies of project reports contact:

Great Lakes 2000 Cleanup Fund
Environment Canada
P.O. Box 5050, Burlington, ONTARIO L7R 4A6
Tel: (905) 336-6276 Fax: (905) 336-6272

Ontario Ministry of Environment and Energy

The Ontario Ministry of Environment and Energy supports the development, demonstration, and implementation of innovative technologies which maintain and enhance the natural environment. To date, the Great Lakes 2000 Cleanup Fund and the Municipal Assistance Program have provided $12 million and $350 million respectively in support of more that 350 projects dealing with water and wastewater facility expansion and upgrades, stormwater management, and wetland rehabilitation. For more information regarding the Water and Wastewater Optimization Initiative contact:

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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EXECUTIVE SUMMARY

The purpose of this guidance manual is to provide sewage treatment plant (STP) managers a comprehensive overview of several optimization tools which had been successfully used to priorize plants for optimization studies, improve effluent quality and defer or minimize capital expansion. Information contained in this manual is based on results, observations and experience gained from a multi-year and multi-facet optimization demonstration program sponsored by Environment Canada and the Ontario Ministry of Environment and Energy (MOEE) in cooperation with other agencies and organizations. Documented benefits from the demonstration program include:

- identification of plants and limiting unit processes which require upgrading and/or expansion before non-compliance occurs;
- improved environmental protection by enabling plants to consistently achieve effluent limits compliance and reduce bypasses;
- effective utilization of existing facilities, often beyond nominal design capacity specified in the Certificates of Approval, thereby eliminating, deferring or minimizing the need for capital expansion;
- determination of the most economical means for upgrading or expanding plants to meet future growth or more restrictive effluent requirements; and
- maintenance of improvements made during optimization studies in the long run, through empowerment of plant operators.

The following optimization tools are presented in the manual.

Self-Assessment Report. This report is designed to provide an uniform approach to assessing the STP's condition, effluent quality and capacity for the calendar year under review, and its ability to comply with effluent limits in the next few years. The report is therefore, useful as a summary report for plant managers and municipal administrators, as well as a screening tool to identify and priorize plants for optimization studies. Pilot testing at 31 STPs indicates that no major difficulties were encountered by operators in completing the report, and 94% of the participants found that the report is useful to identify problem areas in the plant.

Composite Correction Program (CPP). This two-step program identifies and resolves the causes/problems that lead to poor plant performance. The first step known as **Comprehensive Performance Evaluation (CPE)** evaluates the design, operation, maintenance and administration of a STP. Based on study findings, the plant is rated as being capable, marginal or not capable in terms of its ability to meet the required effluent quality with existing unit processes at current flows, as well as identifies and priorizes the performance limiting factors. For capable or marginal plants, the second step known as **Comprehensive Technical Assistance (CTA)** is carried out to resolve the performance limiting factors in a systematic manner. In these cases, poor performance can be due to a combination of factors such as poor wastewater treatment knowledge, motivation, communication amongst and between operations and management staff, inappropriate or out-dated organizational policies and practices. CTA facilitators must work closely with plant operators and managers to develop process control activities and to transfer skills and knowledge at the same time. For plants that are rated as not capable, other optimization tools such as process audit are to be used to identify the most cost effective means to upgrade and/or expand the plant. The first step, CPE can be completed in about five weeks, with one week of on-site activities. The cost for conducting a CPE study range between $5,000 and $20,000. The second step, CTA requires a period of 6 to 18 months to complete, at a cost ranging between $10,000 and $100,000, dependant on the size of the plant, and the number and complexity of the performance limiting factors.

Process Audit. This optimization tool is used when there is a need for more capacity to meet future growth, to meet more stringent effluent limits at flows above the current rate and for those facilities identified by CPE to be incapable of meeting compliance limits at the current flow due to major design deficiencies. Process Audit is a systematic approach used to defining the "ultimate" capacity of existing sewage treatment plant under good process operating and control conditions. The same information can also be used to prepare for the design of the expanded facilities based on design criteria that are less conservative than the MOE design guidelines. Process Audit studies are typically conducted by process specialists, and should cover the critical seasons, for example, winter when biological treatment is the least efficient and spring and/or fall seasons where significant wet weather flows exist. Depending on plant size, study objectives and duration, study costs range between $100,000 and $500,000.

To maintain the improvements made during the optimization studies in the long run, plant operating authorities must ensure organizational policies and practices are updated and adequate to: ensure all staff are committed to consistently achieving effluent limits compliance in the most economical ways; encourage and support staff to acquire and practice new knowledge and skills in technical, inter-personal and management areas. Training should be treated as a process rather than an event. Wherever possible, short periods of on-site training interspersed with telephone consultation provided by trainers/process specialists should be used for operators training.

1. INTRODUCTION

In today's climate of fiscal restraint, managers of municipal sewage treatment plants (STPs) are compelled to "do better with less" by tapping the full capacity of existing facilities. In carrying out this task, managers should ask themselves, their staff and consultants the following questions:

- What are the specific objectives for the optimization study(ies) e.g. increased capacity, improved effluent quality, minimize operation and maintenance costs?

- What optimization approaches are available and proven to be successful?

- Which optimization tool(s) would be best suited to achieve the specific objectives of the study?

- Can major process upgrading or facility expansion be avoided or minimized?

- How can optimized performance be sustained long after optimization study(ies) is completed?

This **Manager's Guide to Optimization** is prepared to assist managers to answer the above questions. The guide presents an overview of several optimization tools which have been successfully applied in Ontario STPs, and some institutional issues that must be addressed in order to facilitate and maintain optimized performance in the long run. Institutional issues are usually related to organizational structures, practices and cultures, and efficient/effective use of human resources.

This guidance manual is based on results, observations and experiences cumulated from a multi-year and multi-facet optimization demonstration program sponsored by Environment Canada, Ontario Ministry of Environment and Energy (MOEE), municipalities and agencies. Documented benefits of optimization studies include:

- identification of plants and limiting unit processes which require upgrading and/or expansion before non-compliance occurs;

- improved environmental protection by enabling plants to consistently achieve effluent compliance and reducing plant bypassing;

- effective utilization of existing facilities, often beyond nominal design capabilities, thereby eliminating, deferring or minimizing plant expansion needs;

- determination of the most economical means for upgrading or expanding plants to meet future growth or more restrictive effluent limit requirements; and

- maintenance of improvements made during the optimization studies over the long run, through empowerment of operators.

2. FUNDAMENTALS

The primary goal of STP manager is to achieve "a good, economical effluent" which in this guidance manual is defined as:

- effluent concentrations/loadings are consistently in compliance with the limits specified in the plant's Certificate of Approval;

- plant bypassing is minimized or eliminated; and

- achieving compliance and bypass reductions in the most cost effective manner by making efficient use of staff, chemicals, energy, and treatment processes.

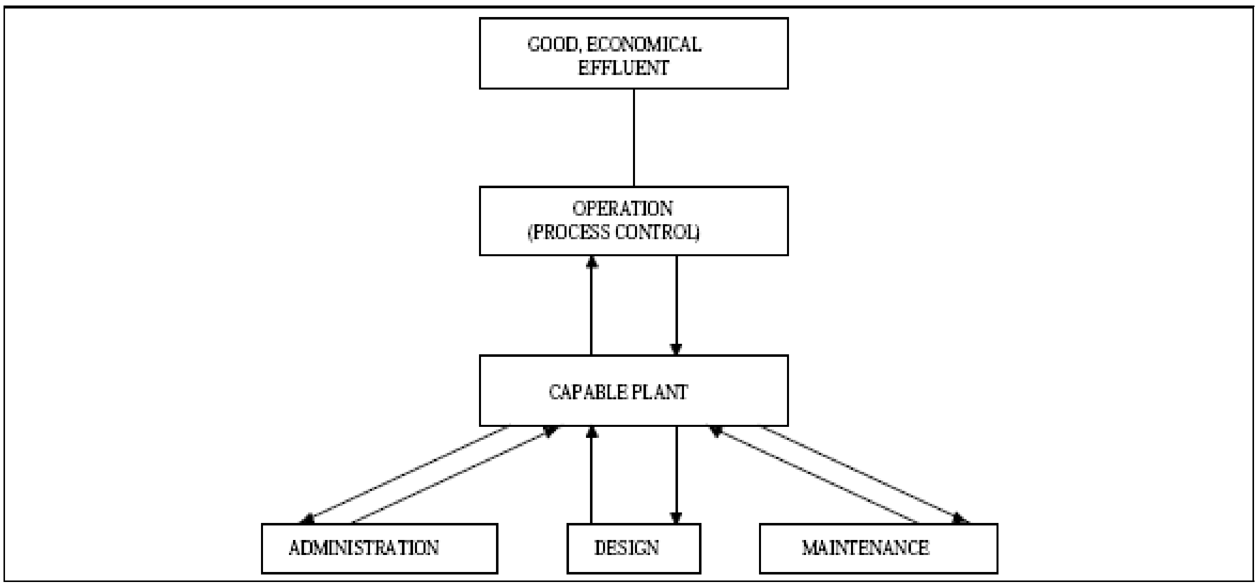

Figure 1 - Relationship of Major STP Components to Achieving a Good, Economical Effluent.

Figure 1 depicts the four major elements: administration, design, maintenance and operation which establish the ability of an existing facility to achieve a good, economical effluent. Administration, design, and maintenance determine a plant's capability to consistently achieve effluent limits (concentrations and/or loadings) at current flows and loadings. Plant adminstration or management helps define a plant's capability by:

- providing an adequate operations and maintenance (O&M) budget;

- hiring, training and motivating staff to maintain and operate the facility; and

- establishing effective organizational policies and practices, for instance, ensuring adequate staff allocation so that process control adjustments are made in an appropriate and timely manner.

Plant design defines a plant's capability by:

- furnishing tanks which are appropriately sized and equipped to successfully treat current flows and loadings; and

- providing process flexibility and controllability so that process measurement and adjustment can be easily made, over a range of flows and loadings conditions.

Plant maintenance program contributes by:

- establishing practices which prevent process upsets and equipment from breaking down; and

- providing for quick repair when equipment fails.

If a plant is capable, it is the responsibility of the operations staff to apply process control on a regular basis to achieve a good, economical effluent. Key elements in process control activities include:

- monitoring and testing of influent, unit process effluent, final effluent and bypass to accurately establish data required for process control and the performance of the facility; and

- applying wastewater treatment knowledge to ensure that monitoring data are efficiently used for process control and in a timely manner.

The performance of many existing facilities can be improved economically by systematically identifying and correcting deficiencies in administration, operations and maintenance. A study (XCG Consultants, 1992) was commissioned by MOE and Environment Canada to assess the most critical factors that led to poor performance at 12 STPs in Ontario. The study also reviewed findings of detailed process studies conducted at 7 other Ontario STPs. Of the top ten performance limiting factors determined by the study, five are in the categories of poor operations and administration. The top ten factors in a descending order of significance are:

- Inadequate sludge wastage and disposal (Operation).
- A general lack of understanding of the fundamentals of sewage treatment processes and the inadequate application of these concepts to process control (Operation).
- Inadequate plant administrative policies and lack of support provided to plant operations staff (Administration).
- Excessive hydraulic loading (Design).
- Inadequate instrumentation and control (Design).
- Extraneous flow due to infiltration and inflow into the sewage collection system (Design).
- Inadequate process monitoring to support process control decisions (Operation).
- Inadequate oxygen transfer capacity to meet oxygen demand (Design).
- Industrial discharge to the STP (Design).
- Inadequate O&M manual (Operation).

These conclusions are in agreement with earlier studies conducted in Ontario (Environment Canada, 1976) and in the U.S. (Gray, A.C Jr., *et al*., 1979; Hegg, B.A., *et al*., 1979; Hegg, B.A., 1980). Improving an organization's policies and practices and developing its human resources are therefore, critical components of optimization studies and to ensure that improvements can be sustained over the long term.

Figure 2 shows that effective optimization of existing facilities consists of four major tasks:

- Task 1: identifying facilities which should be optimized;
- Task 2: identifying and priorizing the major deficiencies preventing the facility from achieving a good, economical effluent or accommodating future growth;
- Task 3: applying cost-effective approaches to resolve deficiencies; and
- Task 4: sustaining improved performance over the long-term by upgrading staff skills and improving organizational policies and practices.

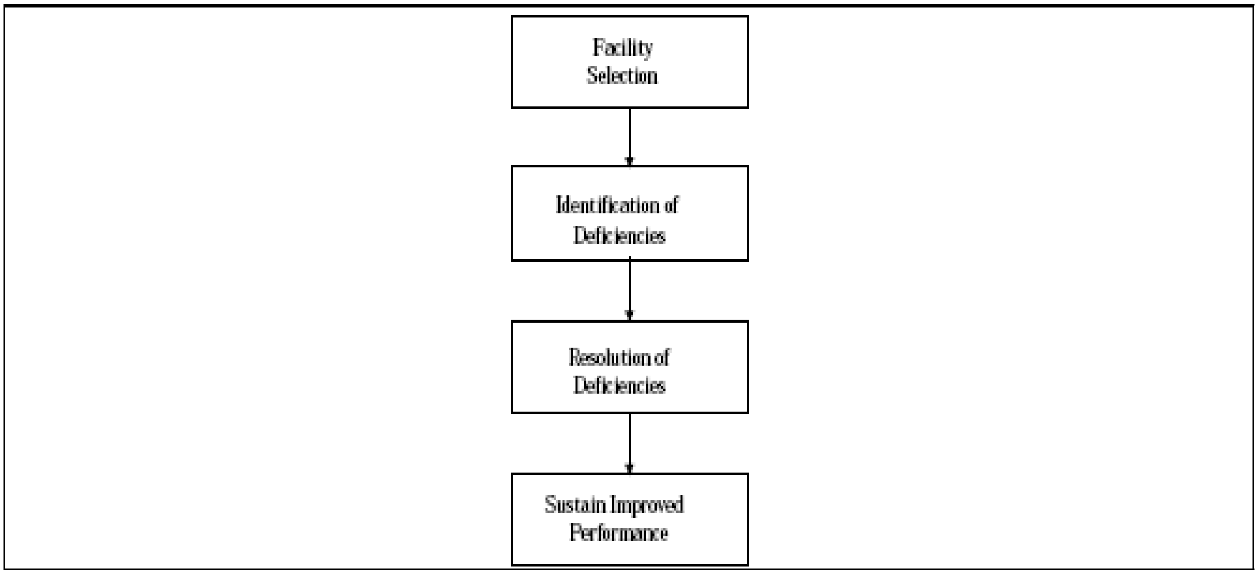

Figure 2 Relationship of Major STP Components to Achieving a Good, Economical Effluent.

Self-Assessment Reporting, the Composite Correction Program, and the Process Audit have been successfully demonstrated in the optimization demonstration program at many STPs for addressing Tasks 1 to 3. They are discussed in Section 3. Issues related to managing and sustaining (Task 4) improvements over the long term are discussed in Section 4.

3. TECHNICAL TOOLS

3.1 Self-Assessment Report

3.1.1 Description

Self-Assessment Report was developed for municipalities to evaluate the current STP performance, and to identify and priorize plants for optimization studies. The report should be completed on an annual basis by the STP operations staff and reviewed with the municipal council and/or senior administrators.

The Self-Assessment Report is divided into eight sections which elicit information on the condition, quality, and capacity of the treatment system using information from January 1 to December 31 of the year. The eight sections are designed to evaluate the status of:

- effluent compliance and plant performance;
- plant capacity - current and five-year projection;
- combined sewer overflows and plant bypasses;

- sludge handling, storage and disposal. The evaluation includes a sludge accountability analysis which compares actual to predicted sludge production rates. This evaluation is useful to verify the accuracy of sludge, influent and effluent quality/quantity data reported by the STP;

- effluent sampling/analysis;

- equipment maintenance;

- operator training and certification; and

- budgets for current operation and maintenance, as well as for future facility replacement and/or growth.

Points are assigned to each piece of information evaluated and a points total is generated for each section. Based on the total points for each section, and/or the report, the STP will fall into one of three categories:

Voluntary: No major deficiency is identified. The owner may wish to implement steps to address minor problems or made further improvements to the present operations;

Recommended: Some deficiencies are identified and the owner is recommended to address the identified problems;

Action: Some major deficiencies are identified which are/will directly affect the STP's ability to achieve and maintain compliance in the future. The owner should conduct a more detailed site investigation to accurately define the problems, determine the causes of the problems and develop a remedial action plan to mitigate the identified causes, before non-compliance actually occurs.

3.1.2 Application

Self-Assessment report can be used as:

- an annual management summary/review report from plant operations staff to head office administration and municipal council to identify current compliance status and resource needs for coming years;

- an annual report required by the Certificate of Approval (subject to agreement with MOE District Office); and

- a report to identify and priorize plant(s) for optimization studies.

3.1.3 Benefits

The report is modelled after a similar report which has been successfully used in the State of Wisconsin for many years. A U.S. EPA funded study (ICF Incorporated, 1991) concludes that the report has improved the communication between the STP operators, managers, municipal councils and the State Department and increasing their awareness of the STPs' current compliance status, deficiencies and resource needs to maintain compliance and efficient operations in the future. The Self-Assessment Report has been credited to be one of the initiatives introduced by the Wisconsin Department of Natural Resources to improve the annual STP compliance rate from about 80% to 98%.

3.1.4 Case Study

The MOEE and Municipal Engineers Association (MEA) pilot tested the Self-Assessment Report at 31 STPs (MOEE, 1994 a). The salient results of the pilot test are as follows:

- 77% of the plants completed the report in 2 person-days (ranged between one and seven days);
- on average, 20 minutes were required to review the report (ranged between 10 and 40 minutes);
- no major difficulties were encountered by the operators/managers in completing the report. Only 9% of the respondents reported difficulties in completing the report; and
- the report was accurate in identifying deficiencies in the current STP design, operations or management. Six percent of the respondents reported that the report did accurately identify the problems that existed in the plant.

3.2 <u>Composite Correction Program (CCP)</u>

3.2.1 Description

The Composite Correction Program (CCP) is a two-step approach to cost effectively improve the performance of STPs. Figure 3 provides an overview of the CCP.

3.2.1.1 Comprehensive Performance Evaluation (CPE)

The first step is known as Comprehensive Performance Evaluation (CPE). Once a manager identifies there is a need to improve the plant performance, CPE can be initiated to more accurately determine the nature of the problems and priorize their causes. The evaluation focuses on four major areas: plant design, operation, maintenance and administration. Attention is paid to both the evaluation of technical data as well as human and organizational factors such as operators' knowledge with process operation/control, motivation, communication amongst and between staff and management, and the resources provided to the plant. The CPE involves the following major steps:

- an initial meeting with plant operations and management staff to explain the objectives of the evaluation and to gain staff's trust and cooperation;

- a critical verification and review of historical plant performance/operational data, and evaluation of the plant design against design standards;

- interviewing both operations and management staff individually or in groups;

- assess the information collected to define the nature of the problems, and priorize their root causes;

- an exit meeting(s) with operations and management staff to present results and discuss recommendations; and

- submission of a written report.

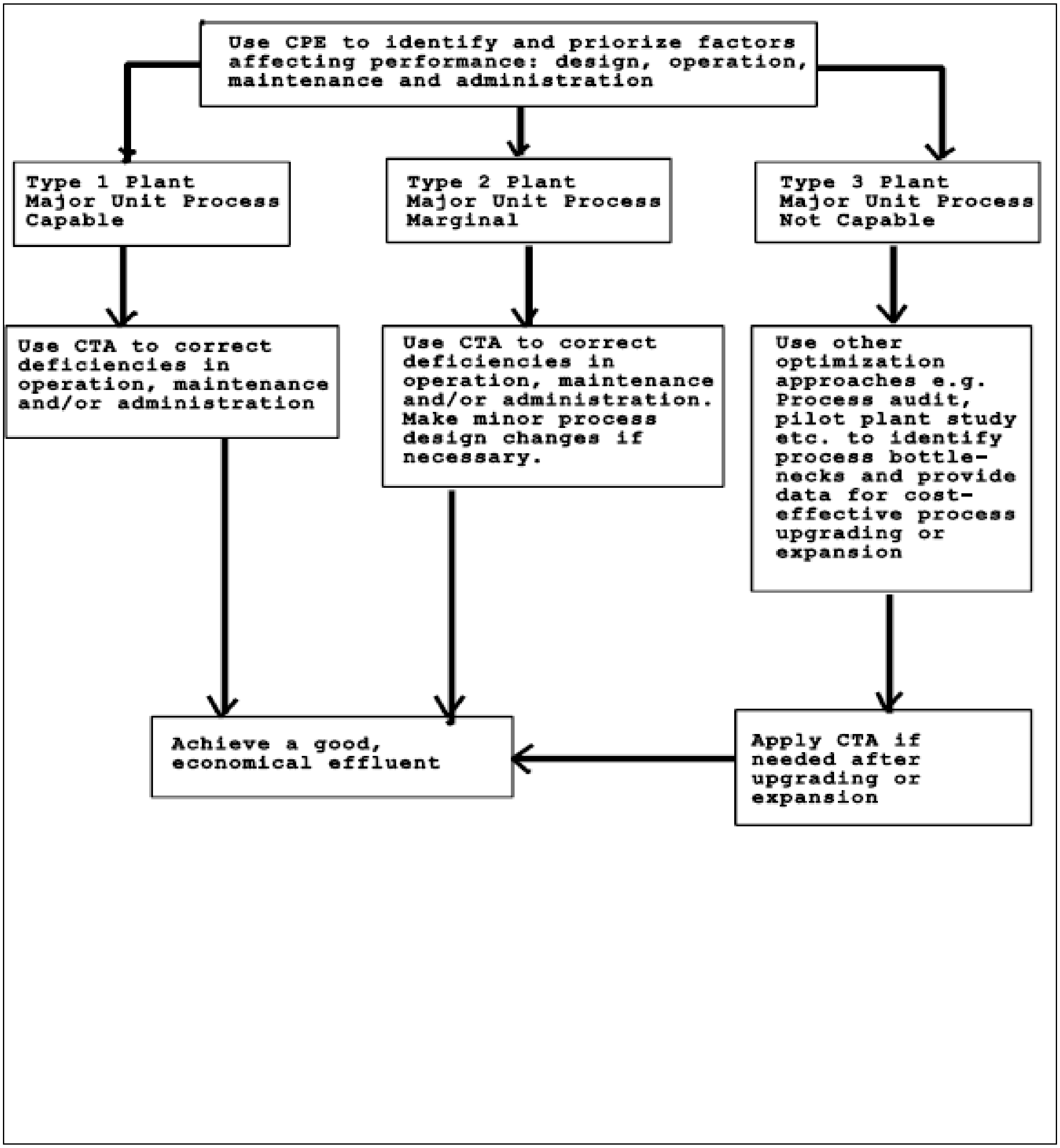

Figure 3 Overview of Composite Correction Program

Based on the results from the major unit process evaluation, the STP is classified as capable (Type 1), marginal (Type 2) or not capable (Type 3), in terms of its ability to achieve compliance at its "current flow". As illustrated in Figure 4, unit process evaluation results are displayed as a performance potential graph. The horizontal bars depict the estimated capacity of each process and vertical lines depict current and nominal design flows. Causes of the problems are identified and grouped into three priority categories:

- *Priority A:* are factors having a major effect on plant performance on a continuous basis;
- *Priority B:* are factors having a major effect on plant performance on a periodic basis, or a minor effect on a continuous basis; and
- *Priority C:* are factors having a minor effect on plant performance.

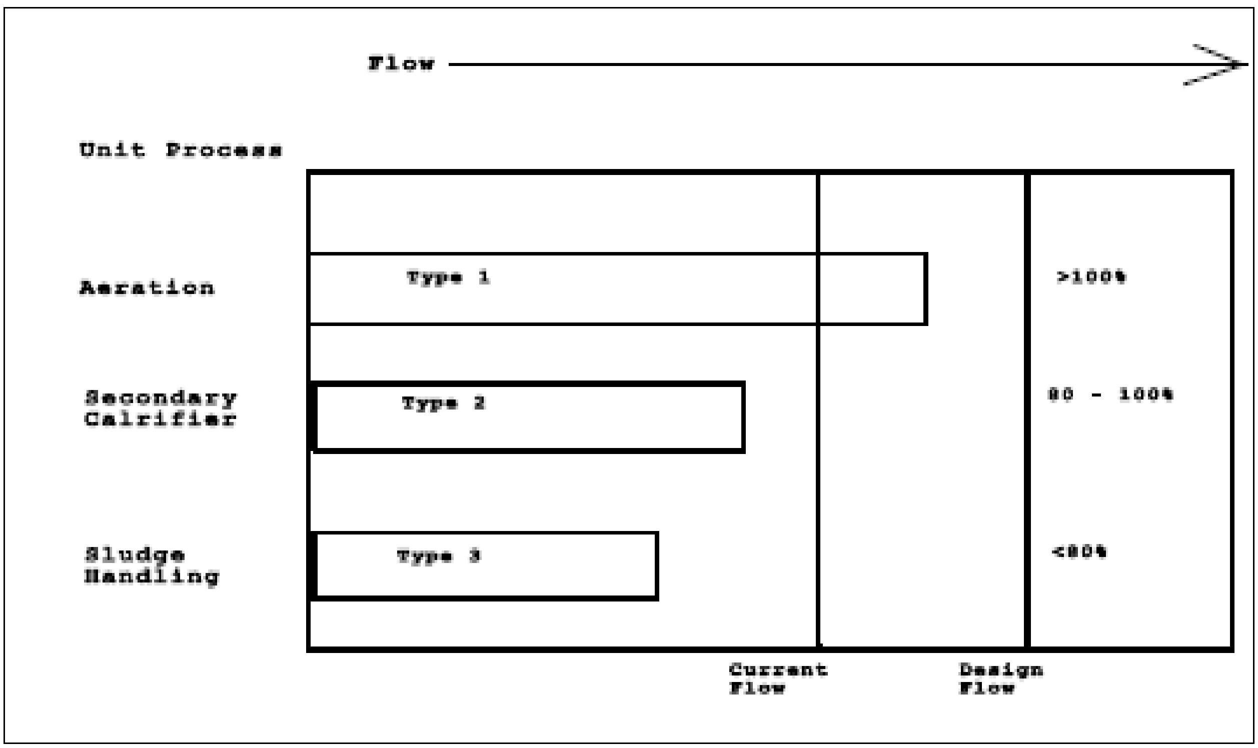

Figure 4 Conceptual Performance Potential Graph

Table 1 is an example of factors that have been priorized in a CPE study.

Table 1 Example List of Performance Limiting Factors

Priority Rating	Performance Limiting Factor	Area
A	Performance Monitoring	Operations
A	Familiarity with Plant Needs	Administrative
A	Wastewater Treatment Understanding	Operations
A	Sludge Treatment	Design
B	Process Control Equipment	Design
B	Process Controllability	Design
B	Secondary Clarifier	Design
C	Alarm Systems Preliminary Treatment Preventative Maintenance Work Environment	Design Design Maintenance Administration

NOTES:

A: Factors having a major effect on plant performance on a continuous basis.

B: Factors having a major effect on plant performance on a periodic basis or a minor effect on a continuous basis.

C: Factors having a minor effect on plant performance.

A CPE study can be completed in one to two weeks, with intensive on-site activities. A written report is submitted within 4 to 5 weeks after the exit meeting(s). It is advisable to have more than one person to conduct the CPE study, since this evaluation covers both technical and non-technical factors and issues. Consultants with expertise in process studies and management evaluation or staff from other STPs should be used to conduct the CPE study. Where staff are used, they should be given the freedom and authority to scrutinize administrative and management practices and issues.

3.2.1.2 Comprehensive Technical Assistance (CTA)

If the facility is determined to be capable (Type 1) or marginal (Type 2), the second step known as Comprehensive Technical Assistance (CTA) study can be utilized to resolve the performance limiting factors/causes. If the STP is identified to be not capable (Type 3), i.e. there are significant unit process design limitations, then other optimization approaches should be followed to determine the most cost-effective means to upgrade and/or expand the STP.

The objective of a CTA study is to improve the performance of an existing STP by systematically addressing the performance limiting factors/causes identified during the CPE study. It is critical for the CTA facilitation team to work closely and cooperatively with the plant operations and management staff to gain their confidence, trust and cooperation, and to achieve transfer of skills to the staff so that improvements made during the CTA study can be sustained in the long term. CTA study focuses on both process control activities and management practices and policies such as "chain of command", workload distribution, plant coverage, communication amongst and between staff, etc. Minor process modifications may be incorporated and evaluated as part of the CTA study.

CTA facilitators can be consultants or staff, provided the staff have appropriate technical knowledge, familiarity and capability to address management issues, motivate and transfer skills to other staff. A team of two or more people is preferred to a single person, to provide a consistent schedule for site visits and follow-up, and the necessary mix of technical and inter-personal skills.

A period of 6 to 18 months is typically required to complete a CTA study. This length of time is required to modify process and equipment, progressively transfer new skills and develop staff confidence to implement new policies and/or operating procedures, and to allow time for the biological systems to respond to changes.

3.2.2 Application

CPE study can be used to:

- identify and priorize performance limiting factors; and
- provide necessary data to develop Terms of Reference for either a CTA study or other optimization studies e.g. Process Audit study.

CTA study can be used to:

- systematically resolve priorized performance limiting factors in plant operation, equipment maintenance and/or administration;
- implement and test the effectiveness of minor process modifications;
- transfer technical and inter-personal management skills to operations and management staff; and
- facilitate plant staff to document improved procedures and practices as part of plant operating manual.

3.2.3 Benefits

Major benefits which have been derived by CCP studies are:

1. Improved Performance
    a. bring STPs back into compliance without major capital expenditures; and
    b. achieve higher quality effluent.
2. Eliminate, Defer or Minimize STP Expansion/Upgrade
    a. operate facility above nominal design capacity (note: stress testing which is part of the Process Audit study is necessary to obtain re-rating approval from MOE);
    b. implement remedial improvements in place of full expansion; and
    c. "operate around" minor design limitations.
3. Identify/Reduce Operating Costs
    a. minimize chemicals usage, e.g. chemicals for total phosphorus removal, disinfection, etc.; and
    b. reduce sludge haulage costs through improved thickening.
4. Improved Process Operations
    a. improved monitoring of effluent quality;
    b. more effective use of process control parameters for process operation; and
    c. better handling and management of sludge production and utilization.

5. Improved Human Resources and Organizational Practices and Policies

a. establish clear and frequent communications;

b. make written operational guidelines available to all maintenance and operations staff; and

c. empower operators and managers to sustain improvements.

3.2.4 Study Costs

Table 2 shows that the costs for conducting CPE and CTA studies by consultants. The costs vary depending on the size and types of facilities.

Table 2 Typical Costs for Conducting CPE and CTA Studies[a]

Types and Size of Facilities	CPE Study		CTA Study
	Person Days On-Site	Costs 1993 ($)	Costs 1993 ($)
Suspended Growth[b]			
<760 m^3/d (0.2 MIGD)	2	3,000 - 7,000	4,000 - 25,000
760 - 7,600 m^3/d (0.2 - 2.0 MIGD)	5	4,000 - 6,000	7,000 - 65,000
7,600 - 37,850 m^3/d (2 - 10 MIGD)	7	5,000 - 25,000	20,000 - 130,000
Fixed Film[c]			
<18,900 m^3/d (o.5 MIGD)	2	3,000 - 7,000	4,000 - 35,000
18,900 - 37,850 m^3/d (0.5 - 10 MIGD)	5	4,000 - 16,000	7,000 - 105,000

[a] Costs based on contracting consultants.

[b] Includes all variations of activated sludge treatment plants.

[c] Includes trickling filters with both plastic and rock media as well as Rotating Biological Contactors.

3.2.5 Case Study

The Composite Correction Program (CCP) was initially evaluated at three plants in Ontario (MOEE and WTC, 1995). The plants included a conventional activated sludge process with a design flow of 18,100 m^3/day and two package extended aeration plants with design flows of 681 m^3/day and 955 m^3/day, respectively. Study durations at these plants were 16 months, 10 months and 14 months, respectively.

Table 3 illustrates some of the significant improvements made by the operations staff during the CCP study. Staff at all three facilities were able to "operate around" minor design limitations which included poor primary clarifier efficiency, lack of instrumentation to measure waste activated sludge flows and limited on-site sludge storage capacity.

Table 3 Effluent Quality (Concentration) Improvements Observed by Three CTA Demonstration Studies

Plant	$TBOD_5$	TSS	TAN
A	40%	0%	90%
B	70%	59%	46%
C	89%	83%	93%

Table 4 illustrates that sludge haulage costs decreased at two of the three demonstration sites as the operators were able to better concentrate their sludge. In Plant A, due to lack of adequate sludge storage facility, excessive amounts of solids were being accumulated in the secondary treatment train. The excessive solids were washed out during high flow conditions, and caused non-compliance with the plant’s effluent limits. The problem was resolved during the CTA study by obtaining an approval from MOEE to store the sludge at a nearby sludge lagoon owned by the same Regional municipality. This remedial action resulted in 40% increase in sludge disposal cost for the Plant.

Table 4 Sludge Disposal Costs After Initiation of CTA Activities

Plant	Savings in Sludge Disposal Cost
A	-40%
B	56%
C	47%

In addition, the following verbatim statements illustrate the human impact of applying CTA studies:

> "This CTA has been a learning experience from start to finish....." (Utility Manager).

> "I have worked at the MOEE for 10 years. I used to hate getting up in the morning and going to work.... Since the start of the CTA I have learned the impact of good process control at my plant ..... Now I can't wait to get to work... The program works...." (Plant Operator).

> "The CTA activities have changed the outlook of (1) the Facilitation Team, (2) operations staff, and (3) management staff towards their profession. This change was derived as a result of the "empowerment" philosophy which is an inherent aspect of a CTA...." (CTA Facilitator).

3.2.6 Further Information

Additional details on conducting CCPs can be found in **The Ontario Composite Correction Program: Optimization of Sewage Treatment Plants** (MOE, Environment Canada, Water Environment Association of Ontario, 1996).

Other references include **Handbook: Retrofitting POTWs** (Hegg *et al*., 1989), **Handbook for Identification and Correction of Typical Design Deficiencies at Municipal Wastewater Treatment Facilities**, (U.S. EPA, 1982), MOEE (1994 b), MOEE and WTC (1994), and Coburn *et al.* (1993).

3.3 Process Audit Study

3.3.1 Description

Process Audit study is used to assess existing municipal and industrial wastewater treatment plants to identify their hydraulic and process bottlenecks, and to establish the plants' ultimate capacity. Review of historical plant design, performance and operation data, flow metering assessment, online/offline monitoring, aeration system capacity analysis, hydraulic modelling, sludge recycle streams analysis, stress testing, unit process tracer tests and dynamic process modelling and simulation are some of the tools used by Process Audit studies. "Stress testing" which subjects a unit process, for example, a final clarifier to higher hydraulic and/or solids loadings until the final effluent exceeds a pre-determined set of effluent quality objectives, is a major tool used by Process Audit study to determine a plant's ability to treat future loads while forestalling plant expansion when possible. If the results indicate that plant expansion is needed, the information gathered will be critical for optimal design and to support application for Certificates of Approval for designs that are less conservative than the MOE design guidelines.

3.3.2 Application

Process Audit study can be used to:

- identify process bottle-necks and provide data for process upgrading;
- confirm ultimate capacity of unit processes and/or plant capacity and provide data for plant capacity re-rating and/or plant design which is less conservative than the criteria stated in the MOE Design Guidelines; and
- identify opportunities for energy savings.

3.3.3 Benefits

The major benefits that can be derived from a Process Audit study are summarized below:

1. Eliminate, defer or minimize plant expansion/upgrade where possible by:

    a. confirming whether the ultimate capacity is greater than the capacity rated in the Certificate of Approval; and

    b. identifying remedial improvements rather than full expansion.

2. Reduce operating costs and improve effluent quality by:

    a. optimizing energy consumption;

    b. optimizing chemical dosage, for example, chemicals for phosphorus removal, sludge dewatering, effluent disinfection, etc.; and

    c. optimizing human resources requirement through better utilization of on-line monitoring and process control instrumentation.

3.3.4 Study Costs

The duration and cost of conducting a Process Audit study is dependent on factors such as project objectives, scope, duration, location and number of project meetings necessary. For thirteen Process Audit studies conducted by specialized engineering consultants, weekly costs ranged from $3,000 to $16,000 for facilities ranging in size from 10,000 m^3/day to 1,000,000 m^3/day (MOE, Environment Canada, Water Environment Association of Ontario, 1996). These projects varied in duration from six weeks to one year.

3.3.5 Case Study

A Process Audit study of the liquid treatment train was conducted at the Metropolitan Toronto Main STP in 1992 (Nolasco *et al.,* 1994). The objectives were to evaluate the ultimate capacity of the liquid treatment train and to maximize the performance of the existing facilities to achieve proposed effluent criteria. Previous studies had estimated costs over $200 million to achieve ammonia removal while maintaining the existing design hydraulic capacity. The Process Audit study involved an extensive monitoring program over 10 months. Oxygen transfer capacity was measured, secondary clarifiers were evaluated using dye tests, and both primary and secondary clarifiers were "stress tested" under high flow conditions. The results from the study indicated that the capital costs to achieve ammonia removal could be reduced to less than $32 million. It also indicated that improvements in performance could be achieved by operational changes, process enhancements, and aeration tank modification. The actual hydraulic capacity of the plant was estimated to be 10 percent higher than the current rated capacity in the Certificate of Approval.

3.3.6 Further Information

A "**Guidance Manual For Sewage Treatment Plant Process Audits"** was jointly published by the Ministry of Environment and Energy, Environment Canada and Water Environment Association of Ontario (WEAO). The manual gives details on how to conduct process audit studies and interpret study results.

Other useful references include the report prepared by WTC and XCG, 1992.

3.4 Integrated Optimization Program

Integrated optimization program aims to cost-effectively optimize all treatment facilities within an area or region with a single or multi-operating authorities by:

- taking advantage of scale of economy in optimizing a number of plants simultaneously;
- providing opportunities to train a core team of plant staff to become proficient with process control and optimization techniques, as well as with various technical and non-technical issues; the core team is essential to sustaining improved performance in the area or region; and
- facilitating knowledge and skills exchange among staff working at different plants.

In Ontario, integrated optimization program has been pilot tested with a single- and multi-operating authorities.

3.4.1 Single Operating Authority

Integrated optimization program is being tested in the Regional Municipality of Halton and within the Department of National Defence. This section describes the background, approach and status of the pilot testing at Halton Region.

The Regional Municipality of Halton collects, treats and disposes of municipal wastewater from the City of Burlington, Town of Oakville, Town of Milton, and Town of Halton Hills (Georgetown and Acton). The Region owns and operates three conventional activated sludge plants with tertiary treatment which discharge to sensitive receiving streams. In addition, the Region has four activated sludge facilities that discharge to Lake Ontario. One of these facilities discharges to the Hamilton Harbour which is an Area of Concern (AOC). The combined population serviced by these seven plants was 300,000 in 1996. The serviced population is expected to grow to 500,000 by 2011.

The Region, in partnership with MOEE and Environment Canada has sponsored a number of single-site optimization studies in the past. In 1991, a Process Audit was conducted of the liquid train at the Burlington Skyway STP. The study established upgrading and expansion requirements to meet future growth and stringent effluent objectives proposed by the Hamilton Harbour Remedial Action Plan (RAP) (CH2M Hill, 1991). Dual-point chemical addition for improving total phosphorous removal was also successfully demonstrated at the Skyway STP. In 1993, a CPE was performed at the Acton STP. The CPE established that the plant was capable at current flows and that year-round nitrification may be achieved through improved process control.

In 1994, the Region with funding support from MOEE and Environment Canada developed and implemented an integrated optimization program for all of seven plants. The objective is to ensure that the water quality of the receiving streams and Lake Ontario is protected as economic growth occurs by:

- optimizing the STPs to achieve the best effluent quality possible;
- basing decisions and determining program success on measurable results (i.e. monthly average compliance, reduced bypassing, demonstrated economy of operations, avoidance of construction, etc);
- empowering STP operators and managers by effective transfer of skills;
- committing to long-term progressive program development;
- addressing institutional issues such as internal and external

organizational policies and practices which may affect optimization practices; and

- developing and maintaining effective partnerships for program development and delivery.

A four-phase approach is being employed. Key elements are as follows.

The Region identified a five member Core Team to undergo training in optimization techniques and to address institutional issues identified during the process. The Core Team consists of two members from Special Projects Section in the Head Office, three from operations (two operators and one supervisor) and one from the regional laboratory. Training was provided by a joint MOEE/Wastewater Technology Center (WTC) Facilitation Team.

Phase 1: Prioritization

The Self-Assessment Report, supplemented by site visits and basic design information, was used to priorize the seven plants for follow-up CPE.

Phase 2: CPE

The Core Team received "hands-on" training in conducting CPEs at three facilities: Milton, Burlington Skyway and Oakville S.E. plants. At the first CPE, the Facilitation Team led the evaluation with the Core Team observing and assisting. At the second CPE, evaluation was jointly conducted by the Facilitation and Core Teams. At the third plant, the Core Team conducted the CPE with the Facilitation Team in an observer role. Regional managers participated in the CPE interviews and attended each of the three CPE exit meetings.

Phase 3: CTA

Two facilities were selected for CTA studies. At the Acton STP, a CTA has been applied to evaluate and demonstrate the facility's ability to achieve year-round nitrification. At the Burlington Skyway STP, a CTA was conducted to address the operations, administration and minor design factors identified during the CPE. CTA facilitation was provided by members from the Core and the Facilitation Teams. In addition, monthly Core Team meetings were held to review the status and approach of the CTA studies.

Phase 4: Maintenance

Following the completion of the CTA studies, the Core and Facilitation Teams will identify strategies to ensure that optimization efforts are sustained within the Region with trained staff and adequate financial resources, without further participation by the Facilitation Team.

3.4.2 Multi-Operating Authorities

In 1994, a pilot program was initiated to demonstrate the integrated optimization program at the Severn Sound AOC. There are seven STPs in the Severn Sound AOC, operated by four different operating authorities: Midland STP (operated by the Town of Midland), two STPs in Penetanguishene (operated by the Town of Penetanguishene, the Mental Health Center in Penetanguishene (operated by the Provincial Ministry of Health), the Elmvale, Coldwater and Victoria Harbour STPs (operated by the Ontario Clean water Agency). The study objective is to develop a viable self-sustaining program including transfer of skills to operations and management staff to optimize the seven STPs within the area to achieve RAP effluent objectives. The RAP effluent objectives are more stringent than their present limits.

The Severn Sound Core Team comprised operators from different STPs within the area. Following priorization, the Core and the MOEE/WTC Facilitation Teams conducted CPEs at the Mental Health Centre and Coldwater STPs. The Core Team identified that the CPE studies were a valuable learning experience and provided knowledge and insight which could be used at their own facilities. The CPE studies concluded that the Mental Health Centre and Coldwater STPs are potentially capable of meeting RAP objectives without major construction, and CTA studies were conducted at both facilities. "Implementation training" was conducted in conjunction with the CTA at the Mental Health Centre (MHC). Staff from the other plants have been trained on mass control techniques at MHC and challenged with implementing these techniques at their own plants. In the last year of the three-year program, monthly meetings of operators from all seven plants were initiated as a mechanism for program maintenance.

Potential benefits of the demonstration program at Severn Sound include:

- demonstrated the ability for Coldwater and MHC STPs to achieve the RAP target effluent total phosphorous limits, without major construction;

- improved operator skills throughout the Severn Sound Area using "implementation training";

- improved communication and cooperation between operating authorities faced with achieving the RAP effluent total phosphorus objectives of 0.1 to 0.3 mg/L;

- developed accurate sludge production values to assist the development of an area-wide sludge management plan; and

- developed and demonstrated approaches and benefits for area-wide optimization program to serve as a model for others areas.

4. SUSTAINING IMPROVEMENTS MADE BY OPTIMIZATION STUDIES

Many STPs can be successfully optimized using the technical tools described in Section 3. However, management support and cooperation by operations staff are essential to sustain improvements made by optimization studies.

4.1 Common Issues

Managers should recognize that non-technical issues such as organization/human resource policies and practices can have as much impact on plant performance as technical issues such as process design and control procedures. The following sections give a brief overview of some of the technical and non-technical issues observed during some of the optimization demonstration studies and how they have affected the optimization efforts. Recommendations to resolve these issues are provided, where appropriate.

4.1.1 Lack of Appropriate Focus

The purpose of a municipal STP is to reliably maintain effluent limits compliance. It was observed during the optimization demonstration studies that some operations and management staff were not aware of their plants' compliance limits specified in the Certificates of Approval. Consistent process control was secondary, while equipment maintenance and general house keeping received more attention.

Effluent quality objectives and compliance limits, along with other plant operating objectives should be clearly established, communicated and committed to by management and operations staff.

4.1.2 Delays in Approving Minor Process Modifications

In many cases, minor process modifications can significantly improve ease of operations and/or effluent quality. For example, a MOEE demonstration study recommended that a weir box to be installed at a small plant to measure recycle flows so that the biological treatment process can be better controlled; and at another plant, the study recommended a baffle and channel to be installed to provide for step-feed to reduce bypass during wet weather conditions. In both instances, significant delays in obtaining approval for these modifications were encountered because:

- the plant operating authorities were not aware of the need to improve the performance of their plants;

- both the operations staff and the MOEE staff were not familiar with the proposed modifications; and

- there was poor communication between operations staff and MOEE staff.

To successfully implement minor process modifications, STP managers must maintain regular liaison with MOE staff, especially staff at the MOE local District Office. If possible, meetings with MOE staff should be arranged at the STP. On-site meeting can often eliminate confusions, uncertainties, and difficulties to describe proposed modifications and/or operational changes in writing. When necessary, efforts should be made to closely monitor the status/progress of the approval requested.

4.1.3 Inadequate Sludge Storage and Disposal

When sludge is not removed from the STP for an extended period of time, the solids will accumulate in the liquid treatment train and eventually impact the final effluent quality. Inadequate sludge storage and disposal have been documented to be a major problem affecting the performance of Ontario STPs (XCG and MOEE, 1994). Specific causes leading to this problem include:

- often there was less than 6 months of sludge storage available for plants which utilize their sludge on agricultural lands;

- there was no land approved to receive the sludge;

- there was no contingency plans to deal with poor weather or occasional contamination of sludge; and

- there was a lack of good record keeping and reporting.

It is the responsibility of the operating authority or plant manager to ensure there are sufficient approved lands to receive the biosolids generated by the plant(s), and space for storage during winter and wet weather conditions when land application cannot be carried out. In Ontario, it is recommended that an eight month storage capacity should be provided. Where the storage facility is inadequate, discussions with neighbouring municipalities to share storage facilities and co-manage a biosolids utilization program should be explored. A good industrial sewer use program is essential to minimize the chance for contamination of biosolids.

4.1.4 Lack of Operator Recognition

Operations staff play a key role in controlling the treatment process to achieve a good, economical effluent (see Figure 1). In some cases, acknowledgement of operations staff as a valuable resource is neglected. To achieve optimum performance, managers must create an environment to foster long-term personal relationship with operations staff, acknowledge their value, encourage and support operators to continuously acquire/improve their knowledge and skills both in technical (i.e. process control knowledge) and non-technical areas (i.e. supervision, communications, etc.).

4.2 Skills Transfer

At some STPs, process control decisions were based on insufficient or inaccurate information due to a lack of understanding of process control fundamentals (XCG and MOEE, 1994). In contrast, knowledgable operators at some facilities were able to "operate around" design deficiencies (MOEE, 1994).

Wherever possible, the following recommendations should be followed:

- regularly review and improve, when appropriate, existing operating procedures and human resource policies;
- conduct on-site training so that the knowledge can be more effectively transferred and implemented;
- develop process understanding that allows accurate responses to dynamic changes in loadings and seasons; and
- use "implementation training" approach to allow plant staff to address non-technical as well as technical issues (see Table 6).

Table 5 illustrates the desirable characteristics and attributes of a good operator, derived from a nominal group meeting. These include leadership and management skills. However, leadership and management skills training is often ignored in operator training. "Implementation training" is a useful approach to integrate leadership and management training with technical training. Table 6 presents a summary of some proven techniques used in "implementation training".

Table 5 Attributes of A Good Wastewater Treatment Operator

QUESTION: What are the attributes/characteristics of a wastewater professional?	
RESPONSE	AREA[1]
Background of experience in wastewater treatment technology.	T
Technically or scientifically oriented.	T
Good ability to interpret and apply concepts.	T, L/M
Experience in plant operations and maintenance.	T
Ability to trouble-shoot.	T, L/M
Good interpersonal skills in dealing with people.	L/M
Technical knowledge, process understanding, communicative.	T, L/M
Inquisitive and analytical mind.	T, L/M
Curiosity about process.	L/M
Apply technical training periodically.	T, L/M
Assumes responsibility for continuous improvement in treating wastewater.	L/M
Rational/pragmatic and balances environmental needs with cost.	L/M

Notes:

1. T = Technical Skill
   L/M = Leadership or management skill

Summary of results from a Nominal Group Process, MOEE and Municipal staff, Venture Inn, Burlington, Ontario, April 22, 1993.

Table 6 Implementation Training: Skills and Techniques (after Lattea)

Skill/Techniques	Application
Recording personal revelations in A-Ha's sheets (see Appendix 1)	During a training event, ideas, suggestions, activities for follow-up criticisms, etc. are recorded that represent a personal revelation.
Small Groups	Small groups encourage sharing of experiences, group learning and enable communication skills to be exercised.
Nominal Group Technique	This technique (silent time, discussion, clarification, voting) is used to achieve consensus about concerns or issues which can be turned into goals.
Time Pictures	A technique to enable participants to see how time is spent and schedule activities which implement knowledge transferred during training.
Topic Development Sheet (see Appendix 1)	Participants work cooperatively to define the benefits, possible obstacles, possible solutions and action steps to achieve a goal.
Implementation Plan (see Appendix 1)	A written list of action items or steps identifying the person responsible and the date due.

Short periods of on-site training interspersed with phone consultation and time for plant staff to apply concepts learned is very effective to empower plant staff to achieve and maintain optimized performance. This training approach creates an environment to develop technical skills and provides opportunities to apply the skills at the operator's "own facility". Successful training can be measured by the ability of a plant to maintain optimum economical effluent quality, over a long period of time.

Managers should recognize that "effective transfer of skills is a process, not an event". "Repeated exposure" is often necessary to ensure operators become proficient with new concepts and skills.

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6. GLOSSARY

Bypass	Flows which are diverted at the headworks of the sewage treatment plant to the waterbody; bypass flows may receive partial treatment or be directly sent to the receiving water
CCP	Composite Correction Program, developed by the U.S. Environmental Protection Agency (EPA), to focus on and address noncompliance in sewage treatment facilities; the CCP has two major components, an evaluation phase known as a CPE and the follow-up technical assistance phase known as a CTA
CPE	Comprehensive Performance Evaluation (first phase of the Composite Correction Program) which identifies Performance Limiting Factors (PLFs) in four areas: administration, operations, maintenance, and design
CTA	Comprehensive Technical Assistance (second phase of the Composite Correction Program) which addresses Performance Limiting Factors with a focus on achieving and maintaining improved performance
Effluent	Treated wastewater flowing from a treatment plant
EPA, U.S.	Environmental Protection Agency, U.S.
Good, Economical Effluent	Effluent concentrations/loadings are consistently in compliance with the limits specified in the plant's Certificate of Approval; plant bypassing is minimized or eliminated; and achieving compliance and bypass reductions in a most cost effective manner by making efficient use of staff, chemicals, energy and treatment processes
MISA	Municipal / Industrial Strategy for Abatement
MOE	Ministry of the Environment
MOEE	Ministry of Environment and Energy
PLF	Performance Limiting Factor, a factor identified by the Comprehensive Performance Evaluation as contributing to poor performance
Process Audit	A detailed technical plant design and performance assessment involving on-line monitoring and "stress testing"
Self-Assessment	A report prepared by the operator on an annual basis for review by the municipal councils and/or senior administrators; the report provides a comprehensive overview of the plant's current compliance status, potential deficiencies and is also useful to identify and priorize plants for optimization studies

STP	Sewage Treatment Plant
Stress Testing	Measuring the performance of a unit process under high hydraulic, and/or solids loading conditions as a method for estimating its ultimate capacity
TP	Total phosphorus
TSS	Total suspended solids
$TBOD_5$	Carbonaceous plus nitrogenous biochemical oxygen demand (five-day)

APPENDIX 1

EXAMPLE FORMS USED FOR IMPLEMENTATION TRAINING

A-HA'S

During each training event, certain ideas will come to mind that represent a personal revelation. We call these revelations A-HA's! Please jot down your personal A-Ha's concerning this event and be prepared to present them. NOTE: You will be requested to turn in these ideas at the end of the seminar.

Name:__________________________

Date:________________________

TOPIC DEVELOPMENT SHEET	
TOPIC/ISSUE:	
BENEFITS:	
POSSIBLE CHALLENGES	POSSIBLE SOLUTIONS

ACTION STEPS:*

* Transfer to Implementation Plan

IMPLEMENTATION PLAN

GOAL: NAME:______________________________

BENEFITS: DATE: ______________________________

NO	A-HA/ITEM	ACTION/IMPLEMENTATION PLAN	BY WHOM	BY WHEN