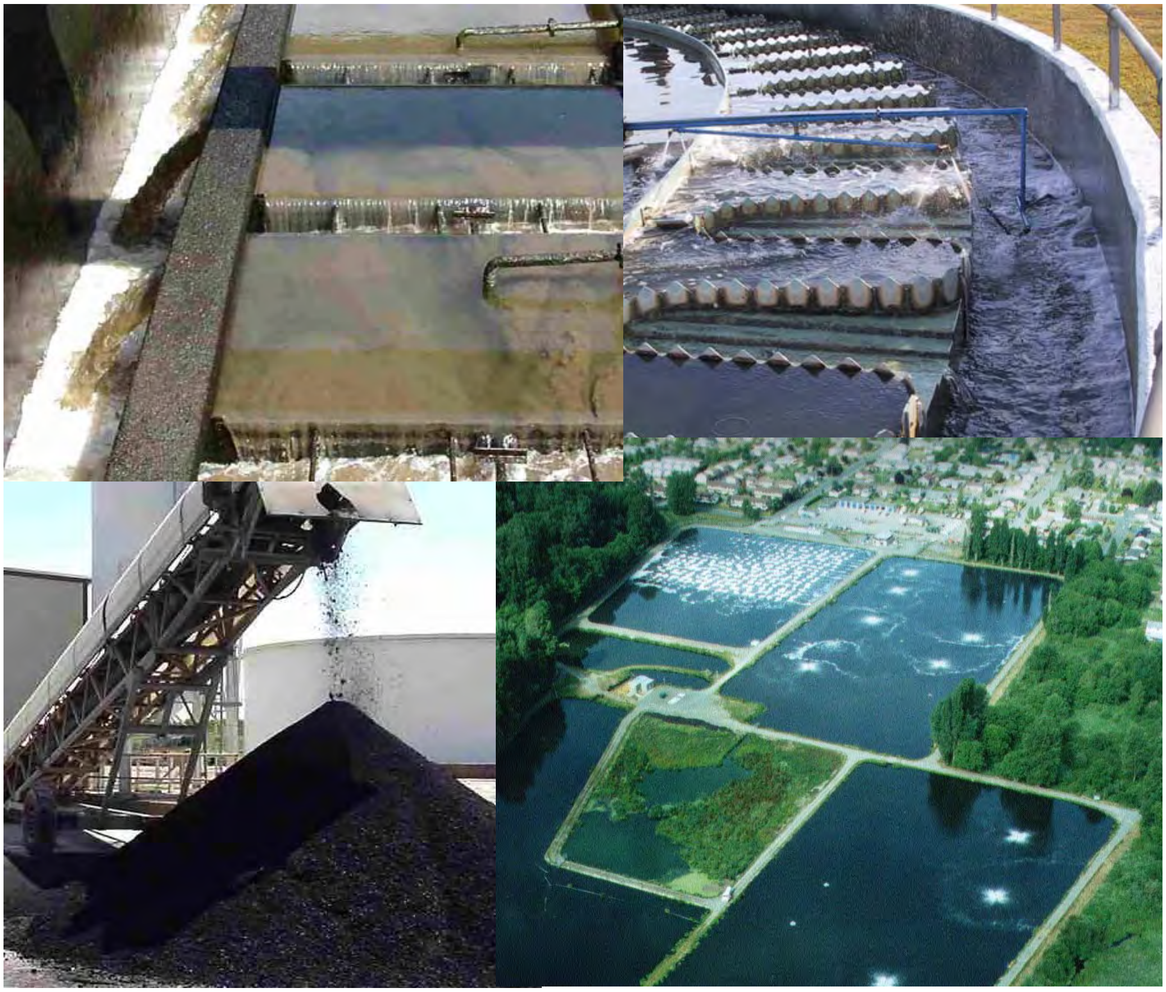

Design Guidelines for Sewage Works 2008

Protecting our environment. Ontario

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Design Guidelines
for
Sewage Works
2008

Ministry of the Environment

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Protecting our environment. Ontario

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HISTORICAL NOTE

Since the establishment of the Ontario Water Resources Commission under the *Ontario Water Resources Act* (1956), the commission engineers used the Ten States Standards for Sewage Works as the reference design guidelines for sanitary engineering practice. These publications were prepared, edited and published, approximately every five years, by the Great Lakes Upper Mississippi River Board of State Public Health Engineers and Great Lakes Board of Public Health Engineers. The commission engineers had also developed and applied internal advisory sewage works design guidelines based primarily on the Ten States Standards and included design, construction and operational experience specific to Ontario.

This practice has continued after the establishment of the Ministry of the Environment in 1973. The Province of Ontario joined the Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers and the Ten States Standards Wastewater Committee in 1977.

Over the years, engineering design criteria based on generally accepted good engineering practice in Ontario have been developed and the following ministry guidelines were published:

- Guidelines for the Design of Sewage Treatment Works (1980, 1984)
- Guidelines for the Design of Sanitary Sewage Systems (1979, 1985)
- Guidelines for the Design of Storm Sewer Systems (1979, 1985)
- Guidelines for Servicing in Areas Subject to Adverse Conditions (1985)

These guidelines have been revised and updated based on Ontario specific engineering practice, the latest Ten States Standards (Recommended Standards for Wastewater Facilities, 2004) and other relevant North American design guidelines and published as the Design Guidelines for Sewage Works (2008).

PREAMBLE

The Ontario Ministry of the Environment's Design Guidelines for Sewage Works is intended for an audience that includes engineers who are responsible for designing sewage works, ministry engineers responsible for reviewing and approving the designs of such works and the municipalities/owners of the sewage works.

It is intended that this Design Guidelines document be used with professional judgment and experience in the design of sewage works and in the engineering review of applications for approval of such systems. The Ministry recognizes that the choice of sewage works designs may be influenced during the planning stages by sustainability issues, such as the cost to design and build sewage works as well as the ongoing cost to operate, maintain, rehabilitate and replace infrastructure.

Designers should note that the ministry has a number of specific guidelines and/or procedures which relate to sewage works that may affect design. Such specific guidelines and procedures take precedence over these Design Guidelines.

Similarly, the use of actual site-specific data is encouraged. Wherever possible, designers are encouraged to use actual data derived from the sewage works monitoring records and characterization studies. Actual data can be compared to the typical values provided in these Design Guidelines for comparison and consideration.

As well, it should be noted that this Design Guidelines document provides design guidance related to established technologies. The fact that other technologies or equipment are not mentioned in the Design Guidelines should not be construed as precluding their use. It is not the intention of the ministry to stifle innovation. The ministry will approve designs of sewage works if the applicant and designer can demonstrate that the works will have a reasonable and substantial chance of success for the particular application. However, design of sewage works using new and innovative technologies and equipment would be approved only where operational reliability and effectiveness of the works has been demonstrated with a suitably-sized prototype operating at its design load in the conditions suitable for the particular application.

Finally, it should be emphasized that this document contains design guidelines. Legislation, including legislated standards and regulations, takes precedence over the Design Guidelines and must be followed. Readers are cautioned to obtain their own legal advice and guidance in this respect.

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CHAPTER 1

LEGISLATIVE FRAMEWORK

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LEGISLATIVE FRAMEWORK

This chapter provides a brief introduction to the acts and regulations which may be applicable to the design of municipal *sewage works* which are defined and regulated by the *Ontario Water Resources Act* (OWRA).

1.1 INTRODUCTION

The designers and proponents of sewage works are responsible for understanding and incorporating all relevant federal and provincial requirements in the planning, design, construction and operation of sewage works and obtaining legal advice with respect to this. It is recommended that designers and proponents be aware of any pending legislative requirements that may impact design considerations. It is essential to confirm any legislative requirements with the most up to date version, as changes occur frequently.

1.2 APPLICABLE LEGISLATION ADMINISTERED BY THE MINISTRY

The *Environmental Assessment Act* (EAA), the *Ontario Water Resources Act* (OWRA), the *Clean Water Act* (CWA), the *Nutrient Management Act* (NMA), the *Environmental Protection Act* (EPA) and the *Environmental Bill of Rights, 1993* (EBR) are statutes administered by the Ontario Ministry of the Environment (*ministry*) that are applicable to municipal sewage works. These statutes can be accessed from the Ontario e-laws website *http://www.e-laws.gov.on.ca* or the ministry website *http://www.ene.gov.on.ca* under the "E-Laws" link.

Municipal undertakings would follow the approved Municipal Engineers Association (MEA) Municipal Class Environmental Assessment (MEA 2007 or most recent version) planning process and thereby meet the requirements of the EAA. For private undertakings that require EAA approval, reference should be made to the *Designation and Exemption – Private Sector Developers Regulation* (O. Reg. 345/93), made under the EAA.

The statutory requirements for approval of sewage works are contained in Section 53 of the OWRA.

The designer or owner should contact the local District Office of the ministry for pre-submission consultation regarding applications for approval of proposed sewage works.

1.3 SEWAGE WORKS REGULATIONS AND SUPPORTING DOCUMENTS

The designer should refer to the regulations under the applicable Acts administered by the ministry as well as the Guidelines and Procedures related to sewage works. Before the design of sewage treatment works can be initiated, the designer needs to determine the effluent quality criteria that the sewage treatment works will need to achieve consistently. Generally, the determination of the effluent criteria will require site specific calculations to ensure consistency with the ministry's Guideline B-1, *Water Management – Policies, Guidelines and Provincial Water Quality Objectives*.

Guidelines with associated Procedures that should be consulted include:

- Guideline F-5, *Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters (1994*):
  - Procedure F-5-1, *Determination of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters*;
  - Procedure F-5-2, *Relaxation of Normal Level of Treatment for Municipal and Private Sewage Works Discharging to Surface Waters*;
  - Procedure F-5-3, *Derivation of Sewage Treatment Works Effluent Requirements for the Incorporation of Effluent Requirements into Certificates of Approval for New or Expanded Sewage Treatment Works*;
  - Procedure F-5-4, *Effluent Disinfection Requirements for Sewage Works Discharging to Surface Waters*; and
  - Procedure F-5-5, *Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems*;
- Guideline F-6, *Sewer and Watermain Installation: Separation Distance Requirements* (1994):
  - Procedure F-6-1, *Procedures to Govern Separation of Sewer and Watermains*;
- Guideline F-8, *Provision and Operation of Phosphorus Removal Facilities at Municipal, Institutional and Private Sewage Treatment Works* (1994):
  - Procedure F-8-1, *Determination of Phosphorus Removal Requirements for Municipal, Institutional and Private Sewage Treatment Works*
- Guideline F-10, *Sampling and Analysis Requirements for Municipal and Private Sewage Treatment Works (Liquid Waste Streams Only)* (1994):

- Procedure F-10-1, *Procedures for Sampling and Analysis Requirements for Municipal and Private Sewage Treatment Works (Liquid Waste Streams Only)*

The designer should ensure that the most current versions of the guidelines and procedures are being used; access is available from the ministry website. The list provided above is for information only and the ministry website *http://www.ene.gov.on.ca/envision/gp/index.htm*, "Forms, Manuals and Guidelines" should be consulted for up-to-date references on currently active procedures and guidelines.

1.4 OTHER APPLICABLE LEGISLATION

Sewage works may be subject to planning-oriented legislation such as the *Planning Act*, the *Municipal Act, 2001*, the *Ontario Municipal Board Act* and others. In addition, it may be necessary to obtain approval from a number of other organizations which have jurisdiction over all or part of the project, primarily involving the Ministry of Labour. Approvals may be necessary from public bodies and authorities such as Ontario Power Generation, municipal plumbing and/or building departments, conservation authorities and the Federal Government (Parks Canada, the Department of Transportation, the Department of Fisheries and Oceans). Liaison with utility providers such as telephone, power and gas companies and railways may also be required. Designers should familiarize themselves with the requirements of all legislation dealing with sewage works, including relevant sections of the *Building Code*, the *Electrical Safety Code*, the *Fire Code* and labour safety regulations. Existing Ontario legislation may be found at the following "E-Laws" website: *http://www.e-laws.gov.on.ca*. Additionally, the *Sustainable Water and Sewage Systems Act, 2002*, is a provincial statute which many municipalities reference when preparing sewage business plans and when considering the economic viability of proposed projects.

1.5 MINISTRY APPROVAL PROGRAM FOR SEWAGE WORKS

The ministry's approvals program is designed so that all undertakings requiring approval under the legislation administered by the ministry are carried out in accordance with that legislation and the ministry's Guidelines and Procedures. The Guidelines and Procedures are intended to provide a consistent approach to various aspects of environmental protection throughout the Province.

The designer of sewage works should consult the newest edition of the ministry document *Guide for Applying for Approval of Municipal and Private Water and Sewage Works*. This document is intended to provide guidance to applicants requesting approval of municipal and private sewage works (other than industrial sewage works) under Section 53 of OWRA. The guide describes the approval process in general, clarifies the information needed to complete the respective application forms, and outlines the technical information that may be required in support of various applications for approval.

There are Ministry Guidelines and related Procedures that cover effluent quality requirements for sewage treatment plants. Designers should consult with the ministry's staff from the Regional Office to determine the effluent quality requirements for specific proposals.

Higher levels of treatment, beyond secondary treatment, may be necessary in some watersheds due to limited assimilation capacity or due to critical downstream uses of the receiving water body. Many *watersheds* in the Province have been designated as requiring higher levels of treatment.

Assuming that a complete and satisfactory application for sewage works is provided and all necessary preconditions (e.g. compliance with the EAA) have been met, the ministry will be able to issue a *Certificate of Approval* (C of A) that will allow construction and operation of the sewage works. In the case of sewage treatment works, the C of A will outline the effluent quality criteria that need to be complied with and the effluent quality objectives to which the sewage works should be designed to, operated and maintained at all times. The effluent non-compliance limits and effluent quality objectives will generally be provided as both concentrations and mass loadings.

1.6 LEGAL CONSIDERATIONS

The designer should determine applicable statutes, regulations, guidelines and procedures for the proposed sewage works and ensure familiarity with the treatment, design and approvals requirements. There is a wide range of legislation that may apply to the planning, design, construction and operation of sewage works. While some are referenced here, no attempt is made for this listing to be complete. The user of this guide should obtain legal advice and understand and abide by any applicable legal requirements.

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CHAPTER 2

PROJECT DESIGN DOCUMENTATION

CHAPTER 2

PROJECT DESIGN DOCUMENTATION

This chapter provides recommendations regarding documentation to support the design and construction of sewage works. The planning and engineering design of *sewage works* can vary with the size and complexity of the undertaking and therefore not all documents listed in this chapter may be relevant for a particular project.

The terms used are consistent with the Professional Engineers Ontario (PEO) Guideline *Engineering Services to Municipalities* (1986; Revised 12/11/98).

The description of technical information and documentation needed to support applications for approval of sewage works are provided in the *ministry's Guide for Applying for Approvals of Municipal and Private Water and Sewage Works.*

2.1 GENERAL

The process of planning and design involves preparation of a number of separate documents in several phases. The number and complexity of the documents depend on the complexity of the sewage works. The planning and design of new sewage treatment plants require the preparation of several reports and many drawings. The design of a sanitary sewer extension may only require preparation of a single engineering drawing with the basis of design and specifications.

A three stage approach, outlined below, identifies the typical planning and design phases involved towards the development of appropriate project design documentation:

Stage 1 – The recommended approach to meet the project objectives is typically determined through a feasibility and pre-design investigation. Normally, Stage 1 will include: an Environmental Assessment (EA) and the preparation of an Environmental Study Report (ESR), a requirement of the *Environmental Assessment Act* (EAA) through the approved Municipal Engineers Association (MEA) Municipal Class EA, feasibility studies, master plans and other special services. The terms of the MEA's Class EA, a planning document approved under the EAA for use in planning municipal sewage works, should be referred to and followed throughout the initial planning process, as and if applicable.

Stage 2 – Preliminary design and reports should include preliminary plans and reports in the form of drawings and documents outlining the nature of the project, a summary of the basis of the engineering design, a preliminary cost estimate, project schedule and a description of the extent of services and recommendations. This is sometimes referred to as the "preliminary engineering report", but should not be confused with pre-design and feasibility studies which are completed in Stage 1.

Stage 3 – Detailed design, final drawings and specifications, should include preparation of: a design brief, final plans (detailed engineering drawings), specifications (for construction, processes, materials and equipment), a final cost estimate, geotechnical and special investigations (e.g., hazardous building material report) and documents required for all approval or permit applications (e.g., permits for construction, approval for waste discharges, stream crossings, air emissions). Detailed engineering drawings include all structural, civil, architectural, mechanical, electrical and *Supervisory Control and Data Acquisition* (SCADA) drawings required to adequately and completely detail the work being proposed to ensure the works are constructed as designed. A report outlining operation and maintenance requirements may also be necessary.

2.2 STAGE 1 DOCUMENTS

Most designs will require feasibility or pre-design investigations. If an environmental assessment (EA) is necessary, it should be completed in accordance with the requirements of the EAA. For projects that do not require an EA, feasibility studies, treatability and pilot studies, pre-design reports and other special services such as the following may be needed:

- Soils investigation;
- Preparation of feasibility studies comparing alternatives in terms of capital, operation and maintenance costs, land requirements, operating efficiency and energy conservation;
- Obtaining topographic plans or photogrammetric mapping; and
- Other special services which may precede the preliminary design and detailed design services described in Stage 2 and Stage 3.

Where the proposed system incorporates processes for which established guidelines are not available, or include equipment and materials where no reliable data from full scale operation are available (e.g., processes that are new or in development - *Section 3.9 - Technology Development*), the following information may also be needed depending on the scope and risks involved in the project:

- All available data pertaining to the proposed process, equipment, or material;
- Results of any testing programs which have been undertaken by independent testing agencies, research foundations and universities;
- Identification of any known full-scale applications of the proposed process/equipment/material, including a description of the type of application and the name and address of the person who could be contacted for technical information on the application;
- Discussion of the impact of the potential failure of the proposed process/equipment/material and identification of the measures proposed

to be undertaken to prevent or remedy any health hazard or non-compliance as a result of such failure; proposed contingencies to modify or replace the proposed process, equipment or material in case of their failure and liabilities associated with the proposal;

- Description of the monitoring, testing and reporting program proposed to be undertaken during the experimental period; and
- The proposed duration of the experimental period.

2.3 STAGE 2 DOCUMENTS

If a preliminary design report is being prepared for the proposed works, it should present the following information, where applicable. If these issues were addressed in an ESR, reference should be made to that document.

- Summary of raw sewage characteristics and design loads;
- Summary of receiving environment investigations and effluent quality criteria;
- Brief description of the proposed facilities including sludge management, where applicable;
- Summary of preliminary design basis, unit operations and process design parameters including information on operational reliability, unit redundancy/back-up (including sludge management facilities);
- Brief description of noise and odour generation potential in context with the separation distance between the sewage treatment plant (STP) and the periphery of the nearest sensitive land-use (buffer zone);
- Description of availability of stand-by power for the STP;
- Documentation of the extent, nature and anticipated population of the area to be serviced, facilities proposed to serve the area and provisions for future expansion of the sewage works to include additional service areas and/or population growth;
- Itemization and discussion of present and future domestic sewage production figures, industrial, commercial and institutional sewage production, infiltration and wet weather inflows used in sizing various components of the sewage collection and/or treatment works;
- Identification of all yard piping including: pipe location, size, depth, material and bedding, suitable inlets and outlets, the design and location of catch-basins, manholes, building connections and other appurtenances;
- Description of all waste streams generated in the STP, including their volumes, composition, proposed treatment and points of discharge;
- Description of the proposed flow metering, sampling and monitoring program, including monitoring of all waste streams;

- Description of the proposed pumping facilities, including the number and capacities of duty and standby pumps and discussion on the ability of the sewage works to treat sewage during power failure events through standby power facilities and/or equalization facilities;
- Brief discussion of the locations of all significant sewage works structures and their proximity to sources of potential water contamination (e.g., lakes, streams, wells) and susceptibility to flooding;
- Consideration and discussion of cost-effective design alternatives in terms of capital and operation and maintenance costs;
- Description of energy efficient systems incorporated into the proposed design to minimize the impact on future energy demands. This should include energy conservation and utilization practices in the selection of process machinery, the location and orientation of structures, use of biogas and the insulation of buildings;
- Identification of suitable procedures and documents for the pre-selection of machinery and equipment;
- Specification of hydraulic grade line;
- Discussion of the design criteria used for proposed sewers including design flows, minimum depth of cover and minimum separation distance from water mains and other utilities;
- Discussion of the planning for any future extensions and/or improvements to the sewage collection and treatment systems;
- Preliminary design plans, all bearing the project title, name of the municipality/owner, name of the development or facility with which the project is associated, name of the design engineer and preparation date and, where applicable, the plan scale, north point, land surveying datum and any municipal boundaries within the area shown. Where pertinent, the following information may need to be provided:
    - General layout and size of existing and proposed sewers and location of major components of other existing and proposed works;
    - General layout (line diagram) of the works (except for sewers);
- *Process Flow Diagrams* (PFDs) for the sewage treatment processes, showing all process components, the direction of flow of all raw and treated sewage, the location of all chemical addition points, the maximum flow of all streams entering and leaving each component of the process and a mass balance for all design parameters around each process component;
- Brief description of any renovations or improvements to the existing structures, sewer rehabilitation and flow modifications; and
- Brief description of buildings and other significant sewage works structures in terms of specific document needs (e.g., *Code for Digester*

Gas and Landfill Installations CAN/CGA-B105 and *Occupational Health and Safety Act* (OHSA)).

2.4 STAGE 3 DOCUMENTS

2.4.1 Design Brief / Basis of Design

A design brief, summarizing the design criteria and presenting the design bases and calculations used in sizing individual components of the sewage works, should be prepared along with final plans and specifications. Where a preliminary report was not prepared or where some parts of the information in the preliminary report are no longer valid or applicable, the design brief should include the applicable information outlined in *Section 2.2 - Stage 1 Documents* as well as the applicable information outlined in the following subsections:

2.4.1.1 Design Brief - Sewers

- Nature and population of the area served (current and design);
- Design Peak Flow;
- Design data and calculations for individual sewers, including the required capacities;
- *Capacity* of the existing (or proposed) sewage works to meet the additional demand; and
- Field investigations.

2.4.1.2 Design Brief - Major Facilities

Major facilities include pumping stations, sewage treatment plants, outfalls and combined sewer overflow (CSO) facilities.

- Basic data on the estimated sewage generation rates from the population and area to be served, including:
    - Design period;
    - Design service population and area and population density;
    - Design industrial, commercial and institutional sewage flows;
    - Wet weather flow; and
    - Design flows (average, peak daily and peak hourly).
- Design flows used in sizing of individual components of the sewage works (outfalls, pumps, channels, treatment process units and storage units, transport and collection facilities);
- Description (types, number and sizes) of all proposed sewage works, process units and equipment, including treatment and disposal facilities and identification of their process design parameters (e.g., screen sizing,

surface overflow rates and retention times in settling tanks, oxygen levels in aeration tanks, chemical feed rates, chlorine concentration and contact time);

- Detailed process and hydraulic design (or sizing) calculations for all facilities, treatment process units and equipment;
- Accurate hydraulic profiles through treatment plants and pumping stations prepared for minimum and maximum flow conditions to a vertical scale adequate to clearly show the elevations of tank tops, channel and trough inverts, weirs and other features directly affecting the hydraulic gradient;
- PFDs showing all process components (i.e., including type, size, pertinent features, rated capacity of process units and major equipment, tanks, reactors, pumps, chemical feeders), direction of flow for all processes, recycle, backwash and waste streams, the location of all points of chemical addition and treated sewage effluent sampling and monitoring, and indicating the minimum and maximum flow rates of all streams entering and leaving each process component as well as a mass balance for all design parameters around each process component;
- Proposed flow metering system, including raw sewage (influent), recycle/return flows, waste flows and treated sewage (effluent);
- Proposed influent sewage and treated sewage effluent quality monitoring program, identification of sampling points, and frequency of sampling and calibration procedures.
- Procedures for calibrating plant instrumentation;
- Proposed system automation and backup procedures;
- Process Narrative; and
- Proposed rated capacity of the new or expanded sewage treatment plant.

2.4.2 Final Plans and Supporting Documents

All final plans should bear the project title, name of the municipality/owner, name of the development or facility with which the project is associated and name of the design engineer, including a signed and dated imprint of Professional Engineer’s seal and, where applicable, the plan scale, north point, land surveying datum and any municipal boundaries within the area shown.

Engineering plans should include plan views, elevations, sections and supplementary views which, together with the specifications and general layout plans, would provide the working information for finalization of the construction contract for the works. These drawings should show dimensions and relative elevations of structures, the location and outline of equipment, location and size of piping, liquid/water levels, location of utilities and ground elevations.

2.4.2.1 Plans of Sewer Systems

General Plan

A comprehensive plan of the existing and proposed components of the sewage works should be prepared for projects involving new sewer systems or substantial additions to existing systems. This plan should show:

- All major topographic features including existing and proposed streets, contour lines at suitable intervals, drainage areas, watercourses, municipal boundaries and land surveying datum used (or assumed bench mark);
- Location and size of existing and proposed sewers and manholes; and
- Location and nature of all existing and proposed components of the sewage works associated with the proposed sewers, including any existing sewer manholes and overflows.

Detailed Engineering Drawings

A detailed plan and profile drawings should be provided for the proposed and adjacent sewers. The profiles should have a horizontal scale of not more than 1:1000 and a vertical scale of not more than 1:100. The plan view should be drawn to a corresponding horizontal scale. Detailed engineering drawings should show:

- Location of streets, sewers and manholes;
- Existing and proposed ground surface, size, material and class of pipe, location of valve chambers, manholes and other appurtenances;
- Location of all known existing structures which might interfere with or affect the proposed sewers, especially watermains, storm sewers and other appurtenances;
- Geotechnical information and groundwater table elevations along the sewer route;
- Details include sewer bedding and anchoring, service connections, bridge crossings, stream crossings, support structures for existing structures in the path of construction, trench bracing, thrust blocks, manhole installations; and
- Any additional descriptive specifications and information, not included in a separate specifications document, required to inform the contractor of all project requirements regarding the type and quality of construction materials and prefabricated components, quality of workmanship, testing of structures and materials to meet design standards and operating tests for the completed works and process components (e.g., leak testing of sewers).

2.4.2.2 Plans of Major Facilities

The major facilities include pumping stations, STPs and sludge storage facilities.

General Plan

A comprehensive plan of the existing and proposed sewage works should be prepared for all projects involving new major sewage works. This plan should show:

- Area to be serviced and the location of the proposed sewage works;
- All major topographic features including, but not limited to, drainage areas, existing and proposed streets, watercourses, contour lines at suitable intervals, municipal boundaries, land surveying datum used (or assumed bench mark); and
- Location and nature of all existing and proposed major components of the sewage works associated with the proposed facilities, including pumping stations, treatment plant, storage facilities, and outfalls together with their individual geo-reference coordinates (Universal Transverse Mercator Easting and Northing).

Site Plans

Individual site plans should be provided for all proposed major facilities of the sewage works and modifications/upgrades of such facilities. Each site plan should show:

- The entire property where the facility is to be or is located, including the property lines and identification of the nature of the adjoining lands;
- Topographic features of the property and adjoining lands, including existing and proposed streets, contour lines at suitable intervals, drainage areas, watercourses, the elevation of the highest known flood level, municipal boundaries and the land surveying datum (or assumed bench mark) used;
- Layout, size and nature of the existing, proposed and future structures on the property showing distances from property lines and show residences and other structures on adjoining properties;
- Test borings and groundwater elevations within site limits;
- Utility routing within site limits;
- Each site plan should be developed according to local municipal bylaws and be approved by the local municipality; and
- Each drawing should bear a seal of a professional engineer licensed in Ontario.

General Layout and Detailed Engineering Drawings

The following general layout and detailed engineering drawings should be provided for all new major facilities of the sewage works and modifications or upgrades of existing major facilities:

- Accurate hydraulic profiles through sewage works such as collection facilities, sewage treatment plants or pumping stations, prepared for minimum and maximum flow conditions to a vertical scale adequate to clearly show the elevations of tank tops (top of concrete), channel and trough inverts, weirs and other features directly affecting the hydraulic gradient;
- General layout plans for all major facilities of the works (e.g., layout of all processes together) including all associated process flow channels and piping (show direction of flow), process and ancillary equipment, air and chemical feed lines, points of chemical addition, and waste streams;
- Construction scale plan and profile drawings (with dimensions and elevations) of all facilities proposed to be constructed or modified, including any additional descriptive specifications and information not included in a separate specifications document.
- All engineering discipline drawings as required to adequately characterize the works need to be provided; and
- Process and instrumentation diagrams (P&ID) showing the inter-connection and operation control arrangements for all process and ancillary equipment and appurtenances.

2.4.2.3 Specifications

Detailed technical specifications should be provided for all sewage works projects. In the case of minor works such as minor sewer extensions, these specifications can generally be noted on the final plans. For more extensive works, separate specifications documents should be prepared.

These specifications should include all construction and installation information not shown on the drawings but required to inform the contractor of all project requirements regarding:

- Type and quality of construction materials and prefabricated components;
- Quality of workmanship;
- Type, size, rating, operating characteristics and quality of mechanical and electrical equipment and installations (e.g., process and ancillary equipment and appurtenances, valves, piping and pipe joints, electrical apparatus, wiring, metering and monitoring equipment, laboratory fixtures and equipment, special tools);
- Type and quality of process materials (e.g., filter media) and chemicals;

- Testing of structures, materials and equipment necessary to meet design standards;
- Margin of error and calibration frequency necessary to meet the performance criteria of effluent monitoring;
- Operating tests for the completed works and process components (e.g. leak testing of sewers and other piping);
- A program for keeping existing sewage works facilities in operation during construction of additional facilities so as to minimize interruption of service;
- Laboratory facilities and equipment;
- The number and design of chemical feeding equipment;
- Procedures for testing, as needed, prior to placing the project in service; and
- Materials or proprietary equipment for facilities including any necessary backflow or backsiphonage protection.

2.5 SEWAGE TREATMENT PLANT PROCESS OPTIMIZATION

The designer may optimize an existing sewage treatment plant to obtain additional capacity or improved performance rather than expanding or physically upgrading the plant. The results of any plant optimization investigation, which forms the basis for any proposed design changes, need to be adequately documented or referenced (*Section 3.10.3 - Process Re-Rating*).

Process optimization needs to ensure that the proposed changes to the existing sewage works will consistently and reliably satisfy the requirements of the C of A.

Process optimization can play an important role in the assessment of an STP's ability to increase its capacity or to meet more stringent effluent quality. There are many sources of information on plant process optimization that can be referenced and four ministry sponsored references include:

- *Guidance Manual for Sewage Treatment Plant Process Audits* (1996);
- *The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants* (1996);
- *Assessment of the Comprehensive Performance Evaluation Technique for Ontario Sewage Treatment Plants* (1994); and
- *Assessment of the Comprehensive Technical Assistance Technique for Ontario Sewage Treatment Plants* (1995).

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CHAPTER 3

GENERAL DESIGN CONSIDERATIONS

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GENERAL DESIGN CONSIDERATIONS

This chapter describes general design considerations related to the construction, operation and maintenance of municipal *sewage works*. Topics covered in this chapter include site selection, security, energy conservation, reliability and odour control. Specific details on design considerations for Pumping Stations and Sewage Treatment Plants are provided in *Chapter 7 - Pumping Stations* and *Chapter 8 - Design Considerations for Sewage Treatment Plants*.

3.1 GENERAL

Sewage pumping and treatment works should be able to handle all flows and loadings to be expected at the works and meet the overall requirements of the works in terms of facility operation, treatment and effluent quality criteria. Designs should anticipate future changes in terms of hydraulic and/or contaminant loadings and provide flexibility in terms of meeting all reasonable expectations.

3.2 DESIGN BASIS

Sewage pumping stations and treatment works need to be designed to meet the current and planned development needs, eliminate *bypasses* and *overflows* (*Section 8.5.6 - Bypasses and Overflows*). Further details on the design basis for facilities and treatment process units are contained in their respective sections.

3.3 SITE SELECTION

Sewage pumping station and sewage treatment plant sites should be located as far as practicable from any existing commercial or residential area or any area that will probably be developed within the plant's design life. The plant site should be separated from adjacent uses by a buffer zone and provided with adequate area for any foreseeable future expansion. Plant outfalls should be placed so as to minimize impacts on public water supply intakes and where applicable, satisfy the requirements of the federal *Navigable Waters Protection Act* (NWA). Information on site selection for sewage treatment plants is included in *Section 8.1 – Sewage Treatment Plant Location*.

3.3.1 Constructability

The design of the sewage works should allow for:

- Practicality/ease of construction;
- A phased approach;
- Maintaining operations during construction; and
- Planning for future additions/expansion.

3.4 OPERATION AND MAINTENANCE

All sewage pumping and treatment works should be designed with consideration to operation and maintenance. All equipment should be able to be serviced both routinely and during breakdown with a minimum of disruption, including provisions for isolating equipment, removing equipment and safety. Critical equipment should be provided with backup capacity to ensure uninterrupted operation of treatment process units and pumping facilities. Special attention should be given to areas that would be designated as confined space areas.

Facilities need to be provided for staff, including personnel, laboratory and maintenance facilities. The facilities required will depend on the size and remoteness of the sewage works.

For a sewage treatment plant, personnel facilities are generally located in the administration building. This building would serve the needs of the supervisory staff, the operation and maintenance personnel and often the laboratory staff.

A sewage treatment plant staffed for eight hours or more each day should possess support facilities for the staff.

The following, in conformance with applicable building codes, should be provided:

- **Washing and changing facilities.** These should include showers, lockers, sinks and toilets sufficient for the entire staff necessary to operate the facility at design conditions. A heated and ventilated mudroom is desirable for changing and storing boots, jackets, gloves and other outdoor garments worn on the job. Each staff member should have separate lockers for street clothes and plant clothes. Separate washing and changing facilities should be available for men and women, with the exception of the mudroom;
- **Eating facilities.** Provide a clean, quiet area with facilities for storage and eating of light meals;
- **Meeting facilities.** Provide a place to assemble the plant staff and visitors. In some cases, the meeting facilities and the eating facilities will be the same; and
- **Supervisors' facilities.** Provide a place where discussions and paper work can be carried out in private.

Small treatment plants that are not staffed 8 hours a day need not contain all of the personnel facilities required for larger plants, but should have a room with a door capable of being locked and contain at minimum a washroom.

Laboratory facilities required will depend on the amount of analytical work conducted onsite. Generally, regulatory samples are sent to an offsite certified laboratory for analysis while process sampling and analysis are done onsite.

Facilities should be provided to allow for adequate maintenance of equipment. Such facilities generally include a maintenance shop, garage, storage space and yard maintenance facilities. Access to nearby municipal garages and other maintenance centres should be considered and duplication of facilities avoided where possible.

Storage space should be provided for spare parts, fuel supplies, oils and lubricants, grounds maintenance equipment and collection system equipment. In larger facilities it may be desirable to have a separate storage building. In smaller facilities it may be desirable to combine storage with the shop or garage so that the stored material can be protected against unauthorized access and use. For design information on storage and handling of chemicals refer to *Chapter 20 - Handling of Chemicals*.

All basements should have a flood alarm system connected to the central alarm system of the facility.

3.5 FLOOD PROTECTION

Sewage pumping stations and treatment plants should be protected against flooding. The treatment process units should be located at an elevation higher than the 100-year flood level or otherwise be adequately protected against 100-year flood damage. Newly constructed plants should remain fully operational during a 100-year flood event. Information on flood protection of sewage pumping stations can be found in *Section 7.1.2 - Flooding* and for sewage treatment plants in *Section 8.1.2 - Flood Protection*.

3.6 SECURITY

Measures should be provided to prevent unauthorized entry of pumping and treatment facilities which may result in personal mishap or disruption of operations. Measures should include:

- Fencing, railings and walls;
- Secured entrance gates;
- Provisions for emergency vehicles (work closely with the local fire department);
- Traffic control signs or signals;
- Provisions for safe transport of chemicals, fuel supplies and sludge; and
- Care should be taken to avoid trapping personnel with these security measures.

Suggested security provisions for remote pumping stations are outlined in *Section 7.6 - Alarm Systems*.

3.7 ENERGY CONSERVATION

The designer should consider the use of energy efficient treatment processes and equipment including motors, blowers and diffusers over the life of the equipment or process. The treatment processes and equipment should be evaluated in terms of life-cycle costing to ensure maximum benefits.

3.8 RELIABILITY AND REDUNDANCY

Reliability and redundancy criteria should be determined to ensure protection of public health and the environment. Standby or redundant capabilities need to be provided for satisfactory operation of the sewage works during power failures, flooding, peak loads, equipment failure and maintenance shutdowns. Generally, sewage pumping stations and treatment works should be designed so that with the largest flow capacity unit out-of-service the hydraulic capacity of the remaining units can handle the design peak instantaneous flow. The designer should ensure that the sewage flow to any treatment process unit out-of-service can be routed to remaining units in service with minimum impact on their performance.

The design of a sewage treatment plant, since it has an effluent discharge into the environment, should be based on the premise that the failure of any single component should not prevent the sewage works from meeting the required effluent quality and quantity criteria, while operating at design flows (i.e., minimum to maximum design flows).

A treatment plant that has strictly defined effluent quality criteria in terms of objectives and non-compliance limits should have a commensurate level of reliability and redundancy of its components. The designer of a sewage treatment plant should consider the following when evaluating and documenting reliability of the proposed treatment components:

- The effluent non-compliance limits and other site specific quality and quantity criteria (i.e., concentrations and loadings) during the full-range of design flows (i.e., minimum to maximum design flows);
- The likelihood of the system having reduced level of treatment or performance;
- The risk to the performance of the system and in turn to the environment, public health and safety if the level of treatment and performance of its components is reduced; and
- The manner and methods by which reliability is provided so that reduced treatment/performance and overflows or bypasses can be eliminated.

As part of the reliability and redundancy analysis of individual process units, equipment or elements, the designer should also consider the definition of the following:

- Critical process element, unit or equipment;
- Critical events:

    - Estimated duration of events;
    - Actions/safeguards; and
- Effect on the effluent quality versus non-compliance limits.

This analysis may be carried out using Table 3.1 as an example.

Such a table should then form a part of the Operations and Maintenance Manual to ensure the long-term satisfactory performance of the sewage treatment system.

3.9 TECHNOLOGY DEVELOPMENT

Implementation of new technologies and re-rating of existing facilities require special considerations. For the purpose of this Design Guidances document, new technology and proven technology are defined below.

3.9.1 New Technology

Any method, process, or equipment proposed to collect, convey, treat or dispose of sewage that is being tested or has been tested at pilot-scale or at full-scale level, but lacks an established performance record.

3.9.2 Proven Technology

A proven technology has an established performance record and means a technology with:

- A minimum of three separate installations, operated at near design *capacity*;
- A minimum of three years of operating record at three separate locations; and
- A minimum of three years operating record showing reliable and consistent compliance with the design performance criteria without major failure of either the process or equipment.

The designer should be aware that new technologies may have a higher risk of failure than proven technologies. The degree of risk of failure can be evaluated through review of sequential stages of a new technology development where the risk of failure is reduced with each of the following sequential steps:

- theoretical concept;
- development at the laboratory or bench-scale;
- experimental stage consisting of pilot-scale program and field application testing;
- extensive pilot or full-scale testing; and
- established performance record.

Table 3-1 - Example of Reliability Analysis of Sewage Treatment Plant Components

Element	Example Events	Event Duration	Frequency	Action/ Safeguard	Effect on Effluent
Pumps	pump failure	Weeks	1 in 5 years	firm capacity standby pump off-peak hours storage	none
Chemical system	components failure pump failure	Minutes Hours	1 in 1 year 1 in 3 years	alarm, analyzer auto switch; backup system spares	minimal minimal
Pre-treatment	equipment or process failure	Weeks	1 in 5 years	firm capacity, backup systems or bypasses	none
Primary clarifier	replace motor or gear box equipment failure	Hours Weeks	1 in 10 years 1 in 5 years	spare parts on site run with unit out of service	minimal minimal
Final clarifier	replace motor or gear box	Hours	1 in 10 years	spare motor on site bypass to other clarifier, if available	some degradation of effluent quality for short period of time
Liquid-train biological processes	equipment failure	Weeks	1 in 5 years	spare parts on site, run with unit out of service	some degradation of effluent quality
Disinfection	equipment failure	Days	1 in 5 years	alarms, spare parts on site	significant impact on effluent quality
Biosolids processes	equipment and process failure	Weeks	1 in 5 years	firm capacity or operate with fewer units	minimal
Power	power outage equipment failure	Days Days	1 in 5 years 1 in 3 years	alternate grid generator	none minimal

A new technology that is considered for a site-specific application as part of an experimental or pilot program should meet the following requirements:

- The size of the principal components and duration of the pilot program should be such that physical, chemical and/or biological processes are accurately simulated under representative conditions;
- Process variables normally expected in full-scale application have been simulated;
- All recycle streams have been considered;
- Variations in influent sewage characteristics that can substantially affect performance in full-scale application have been anticipated and simulated;
- The duration of testing has been adequate to ensure stable operating conditions and subsequent consistent performance has been confirmed;
- The service life of high maintenance or replacement items have been accurately estimated;
- Basic process safety, environmental and health risks have been considered and found to be within reasonable limits;
- Types and amounts of all required process additives have been determined and tested; and
- A contingency plan should be in place in the event that the new technology fails to meet the expected performance.

Designers considering a full-scale application of any new treatment technology should evaluate the data and other information of any testing programs which have been undertaken by independent testing agencies necessary to ensure the viability of the proposed treatment technology and application and document their findings in the Design Brief (*Chapter2 - Project Design Documentation*).

Note that specific new technologies are not discussed in this Design Guidances document.

3.10 SEWAGE TREATMENT PLANT CAPACITY RATING

3.10.1 Design Capacity

In the case of a new municipal sewage treatment plant, a conceptual design capacity will be developed through the Municipal Engineers Association's Municipal Class Environmental Assessment (i.e., MEA's Class EA) process and will be documented in the Environmental Study Report (ESR). Once the ESR has met the requirements of MEA's Class EA, this conceptual design capacity will form the basis of detailed engineering design resulting in plans and specifications which, in turn, will be used for obtaining a *Certificate of Approval* (C of A) from the approving Director at the Ministry of the Environment (*ministry*). This conceptual design capacity should be documented in the ESR and confirmed as the proposed *rated capacity* in the final design brief. The rated capacity is the highest average annual flow during which the sewage treatment plant can consistently meet site specific effluent quality criteria. It will be specified in the C of A as the rated capacity of the approved sewage treatment plant.

In cases where an expansion, alteration or modification is required to an existing treatment plant, the proponent will need to determine the applicable Schedule (Schedule A, A+, B or C) of MEA's Class EA that is relevant to the undertaking. One of the factors in making this determination is the "existing rated capacity" of the "existing sewage treatment plant" referred to in the Schedules included in Appendix 1 of MEA's Class EA document. This "existing rated capacity" is the rated capacity of the sewage treatment plant specified in the existing C of A for the facility. If the proposed undertaking results in flows to the sewage treatment plant that exceed the rated capacity, the expanded flow requirement needs to form the basis of further Municipal Class EA considerations and subsequent detailed engineering design.

3.10.2 Rated Capacity

The rated capacity of a sewage treatment plant, in accordance with the C of A, is defined as the average daily flow which the sewage treatment works have been approved to handle, calculated as the cumulative total sewage flow to the sewage works during a calendar year, divided by the number of days during which sewage was flowing to the sewage treatment works that year. This is equivalent to the Design Average Daily Flow.

In establishing the design capacity of the sewage treatment plant the designer should consider:

- The ability of the sewage treatment plant to consistently produce treated effluent meeting all applicable Ontario regulations, guidelines, procedures and site specific effluent quality criteria;
- The rated capacity should be established by assessing the capacity and performance of each process in isolation and in conjunction with the entire process train operating as a system to identify the process with the lowest treatment capacity that represents the capacity limiting process or the treatment rate of its slowest unit operation that forms the rate controlling step[1]; and

[1] The products of a sequence of processes can be formed no faster than the rate of the slowest step in the sequence. Therefore, if one of the steps in a sequence is slower than all the others, the overall process rate is limited by and is exactly equal to, the rate of this slowest step (the rate controlling step). The rated capacity of the treatment system is therefore based on the rate controlling step under specific operating conditions such as influent sewage quality and flow variations, temperature and effluent quality requirements.

- The flow(s) and loadings into the sewage treatment plant, process components, trains or stages, (the design maximum, minimum, hourly and instantaneous flows for each individual treatment process and the overall plant).

The designer should undertake a detailed design to meet these requirements in the context of the issues addressed in these design guidelines and specifically in *Section 8.5 - Major Design Criteria*. The rated capacity of the sewage treatment plant will be confirmed in the review of the proposed design by the ministry as part of the C of A application review process and will be included in the C of A issued after the review is completed.

The sewage treatment plant rated capacity, as defined in the final design brief and subsequently in the C of A, is based on the design assumptions at the time of applying for approval. Improvements to operational knowledge, technologies, changes in sewage quality characteristics, conditions of the physical facilities, and process optimization can change the performance of an STP and providing a basis for the re-evaluation of the rated capacity of the STP.

3.10.3 Process Re-Rating

Sewage treatment plant re-rating is the practice of evaluating sewage treatment plants or unit treatment processes to determine if it is possible to consistently operate the STP or unit treatment processes at a higher capacity than the original rated capacity. Process unit or STP re-rating requires at least the following:

- Operational data from continuous operation of a full-scale installation treating or conveying the type and strength of sewage to be treated;
- Automatic indicating, recording and totalizing flow-measurement equipment should be provided. Total flow and other process control measurements should be taken and recorded daily or at a frequency required to verify the operation of the facility or unit process at the proposed re-rated capacity; and
- A proper process audit with stress testing should be carried out, with adequate sample collection and analyses, to demonstrate effectiveness and efficiency under minimum and maximum design flow and loading conditions over extended periods of time such that stable operating conditions have been achieved.

In addition, the following specific elements need to be addressed in all proposals for sewage treatment plant re-rating:

- Impacts of the proposed change on the sewage treatment plant ability to reliably and consistently comply with effluent quality criteria and C of A conditions;
- An evaluation of the potential for the treatment system upset, overflow, by-pass or non-compliance with effluent limits;

- An evaluation of re-rating the sewage treatment plant versus expanding of the plant based on the capacity to accommodate new growth. The community's historical and anticipated rate of growth should be considered;
- An evaluation of the impact of re-rating the sewage treatment plant on operation and maintenance of the facilities. This evaluation should, at a minimum, include the impact on treatment plant operators, including level of certification needed and the need for additional process control and monitoring; and
- For more detailed information on sewage treatment plant re-rating and treatment processes optimization, the designer is referred to the following ministry documents:
  - *Guidance Manual for Sewage Treatment Plant Process Audits* (1996);
  - *The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants* (1996);
  - *Assessment of the Comprehensive Performance Evaluation Technique for Ontario Sewage Treatment Plants* (1994); and
  - *Assessment of the Comprehensive Technical Assistance Technique for Ontario Sewage Treatment Plants* (1995).

3.11 EMISSIONS OF CONTAMINANTS TO AIR

For all sources of emission of contaminants to the air at sewage works (e.g. gases from air striping process, exhaust emissions, noise of standby or stationary generators, noise of air blowers or compressors, odours) the requirements of Section 9 of the *Environmental Protection Act* (EPA) need to be satisfied.

The *Air Pollution - Local Air Quality Regulation* (O. Reg. 419/05) made under the EPA specifies the maximum allowable concentration of the specific air contaminants at the *point of impingement*. Compliance is achieved by maintaining the point of impingement concentrations of the contaminants discharged from the source of emission below the maximum concentrations stipulated in the regulation. Typical points of impingement are the property line and all critical receptors such as building air intakes or windows.

3.12 SAMPLING AND MONITORING EQUIPMENT

Sampling devices should be compatible with the needs of the influent and effluent quality monitoring program. The type of sampler and sample container used depends on the parameter being tested in the sample. Sample devices include dippers, vacuum lifts and pumps (peristaltic, positive displacement or centrifugal). The amount of lift should be a design consideration.

Samplers need to maintain a sampling velocity which will prevent the solids in the sample from settling in the sampler lines. Composite samplers should be flow proportional and capable of sampling flow over a 24-hour period. Sampling lines should be large enough to carry suspended matter. A sampler should have a purge cycle to expel any material left in the sample line from the previous sampling. To comply with sample preservation, most samplers will need a means of refrigeration for the sample.

General guidelines to be used for automatic samplers include the following:

- The sampling device should be located near the source being sampled, to prevent sample degradation in the line;
- Long sampling transmission lines should be avoided. If sampling transmission lines are used, they should have velocities sufficient to prevent sedimentation. Provisions should be included to make sample lines removable and easily cleanable. Minimum velocities in sample lines should be 1 m/s (3 ft/s) under all operating conditions;
- Sampler control should consider continuous flow or flushing to ensure a representative sample is taken;
- Samples need to be refrigerated unless the samples are not affected by biological degradation;
- Sampler inlet lines should be located where the flow stream is well mixed and representative of the total flow;
- Sampling access points should be provided for return and recycle lines, sewage inflows and outflows and waste sludge lines; and
- Access to sampling sites should be provided in the design of sewage pumping and treatment facilities to obtain grab samples.

3.13 HYDRAULICS

The designer of new sewage works needs to evaluate the existing and proposed hydraulic grade lines to ensure raw sewage or effluent pumping requirements are met under steady state and peak flow conditions for the design life of the facility. The use of gravity flow, where appropriate, may result in lower capital and operating costs, but generally restricts the siting of the treatment works and may not be suitable for use with some treatment processes. Such factors should be carefully evaluated to determine the best possible hydraulic and siting configuration.

3.13.1 Flow Metering

All sewage pumping stations and sewage treatment plants should have flow measuring devices to measure the sewage flow to the works and through the sewage treatment plant, including overflows and bypasses. In addition, flow through unit processes, backwash flow, chemical and gas flows should be metered for monitoring and controlling the treatment processes (*Chapter 9 - Instrumentation and Control*).

The designer should consider the importance of meter accuracy, specifically as it relates to compliance with C of A effluent quality criteria relating to contaminant loadings.

3.13.2 Flow Distribution

Flow distribution or splitting refers to the separation of a flow stream into two or more smaller streams of a predetermined proportional size. Flow splitting allows unit processes to be used in parallel and applies mainly to liquid streams but can also be used for sludge streams.

3.13.2.1 Flow Splitting Devices

The following devices can be used for flow splitting; many can also be used for flow measurement. These include:

- **Flumes** - Flumes are open channel structures or devices that produce a headwater (upstream) elevation related to a predictable flow going through the structure as long as the flumes are operating in a non-submerged condition. The higher the flow, the higher the headwater elevation. Two or more identical flumes will pass the same flow with the same upstream head. If two or more identical flumes share the same headwater such as in a splitter box, they will effectively split the flow evenly among the flumes. One advantage in using flumes to split the flow is that they can operate accurately with very little available head. Flumes are not recommended if the flow needs to be split unevenly because the flow is not linearly related to the throat width of the flume;
- **Weirs** - Weirs are flat plates set in a channel which, like flumes, produce an upstream head proportional to the flow going over the weir. The main advantage of weirs is that they are fairly compact and inexpensive. Their main disadvantage is that they need sufficient head to operate properly. Generally, the weir plate itself has to be in the order of a minimum of two times the maximum head generated behind the weir in height. If the flow is to be split unevenly, *suppressed weirs*, circular weirs or Cipolletti weirs should be considered;
- **Control Valves** - Control valves are used to split the flow when little or no head is available or space constraints prohibit the use of a splitter box. There are several valves suitable to control flow splitting. Butterfly valves can be used in large-flow situations where the chance of plugging with stringy materials is low. Pinch valves are ideally suited for flow control when there is no debris in the fluid. Plug valves, ball valves and other valves which do not plug are appropriate for flow splitting control. It is best if the valves are automatic and controlled by a flow signal from each of the individual flow paths. In this way the flow can be instantaneously totaled and divided out in a predetermined way; and
- **Symmetry** - Symmetry has been relied on to split flows with mixed results. Symmetrical flow splitting relies on the symmetry of the inlet structures to the upstream flow that is being split. One problem with

reliance on this type of flow scheme is maintaining complete dynamic symmetry throughout the actual design flow range. Small variations in approach velocity, channel and pipe roughness and downstream head losses can have a major impact on the accuracy of the flow split.

3.13.2.2 Factors Affecting Flow Splitting

The following points should be considered in the design of the various components of the hydraulic flow distribution:

Upstream Conditions - If the upstream flow velocity is above 0.3 m/s (1 ft/s) significant velocity head can develop. If the flow is not perfectly symmetrical in relation to the splitting devices, the velocity head can develop uneven pressure head on the different flow splitting devices. This causes an uneven or unintended flow split. A sufficient amount of head has to be available upstream of the splitting devices so as not to cause flooding of the upstream processes.

Inadequate Head or Pressure - If there is insufficient elevation difference between the upstream process and the downstream tanks, the flow splitting devices will not function properly and submergence of the splitting device can occur. When a device is submerged, the tail water depth prevents free fall and an aerated *nappe* from occurring through the device. The head on the device, in this case, is no longer related in a consistent way to the flow going through the device. If one or more of the devices are submerged, but have the same headwater, the devices cannot reliably split the flow in the required ratio. The results would be unpredictable and inconsistent.

Approach Conditions - The flow conditions approaching the splitting devices are critical to the success of the flow splitting effort. The flow velocity in the headwater area should be 0.3 m/s (1 ft/s) or less to minimize any potential velocity head, which is described by the equation $V^2/(2g)$ (where V is velocity and g is acceleration due to gravity). The additional velocity head could turn into pressure head resulting in uneven head loss among the splitting devices, changing the flow split. An uneven approach velocity distribution can also result in an unacceptable change in the flow split.

Downstream Conditions - Downstream conditions can seriously affect the flow splitting capability of splitting weirs. Sufficient head needs to be available between process units to allow the proper functioning of the flow splitting devices. In particular, the flow splitting device needs sufficient free fall to the tail water for it to work properly.

Submerged Flow - Submerged flow occurs when the tail water depth is too high to allow free fall through the splitting device. Without free fall, the splitting device will not work properly. Certain devices such as flumes can tolerate a degree of submergence and still function. Weirs need at least 0.3 m (1 ft) of free fall to allow for an aerated nappe. If a device is overly submerged, the flow through the device is affected by the tail water depth, which alters the flow split design.

Improper Sizing of Primary Device - For satisfactory results, the size of the primary flow splitting device needs to match the flow being divided. If the primary flow splitting device is too large, it will not function properly. A minimum amount of head loss has to be generated through the device: For small flows, at least 0.2 m (0.7 ft) head loss needs to be generated. For larger flows, more head loss is required to split the flow. If the flow over a weir is insufficient it may result in the spillover running down the face of the weir. The nappe is no longer considered aerated and it acts as though it were a submerged flow. This can result in a pulsing of the flow over the weir as the nappe hugs and then separates from the weir. The resulting split flow is unpredictable. If the primary splitting device is too small it will generate too large a head to be accurate. It will also generate excessive head loss which may not be acceptable. Finally, the device would need a higher free fall to function.

3.14 MANUALS & TRAINING

3.14.1 Operations Manual

An Operations Manual should be supplied to the sewage treatment plant as an essential part of the design. The Operations Manual should include detailed descriptions and explanations of the treatment processes and operational strategies for meeting the effluent quality criteria specified in the C of A. All standard operating procedures developed for the plant should be included in the Operations Manual. The manual should be provided in standard electronic format and cover the following topics:

- A plant overview and process control philosophy statement;
- Detailed unit operations and chemical dosing for normal operation and emergency situations;
- Simplified system schematics that take into account the spatial relationships involved;
- Descriptions and operational procedures for facility utilities (e.g. Heating, Ventilating, and Air Conditioning (HVAC), plant service water, security);
- General safety information, including provisions to keep *Material Safety Data Sheets* (MSDSs) up-to-date;
- Spill containment and emergency procedures and reporting;
- Emergency power systems and electrical system operation;
- Security of infrastructure, electronic files and/or programs and response procedures to breaches or intrusions;
- The licensing requirements as indicated in *Licensing of Sewage Works Operators* R*egulation* (O. Reg. 129/04) made under the *Ontario Water Resources Act* and other applicable regulations;

- Monitoring, reporting and documentation procedures;
- Procedures for bringing equipment on-line after maintenance;
- Reliability and redundancy analysis of system components;
- Detailed routine maintenance procedures;
- Alarm notifications and response procedures;
- A list of emergency contacts and locations of contingency plans; and
- A list of major equipment suppliers.

3.14.2 Equipment Manuals

Equipment manuals including parts lists and parts order forms, operator safety procedures and an operational troubleshooting section should be supplied to the owner as part of any proprietary unit installed at the works.

3.14.3 Training

Provisions should be made for operator instructions with documentation at the start-up of any new facility, equipment or process.

3.15 HEALTH AND SAFETY

Consideration needs to be given to the safety of sewage works personnel and visitors. The designer should refer to all applicable codes and regulations under the *Occupational Health and Safety Act*, the *Building Code Act, 1992* and the *Fire Protection and Prevention Act, 1997*. Items to be considered include noise arresters, noise protection, confined space entry, protective equipment and clothing, gas masks, safety showers and eye washes, handrails and guards, ladders, warning signs, smoke detectors, toxic gas detectors and fire extinguishers.

Equipment and chemical suppliers should also be contacted regarding particular hazards of their products and the appropriate steps taken in the facility design to ensure safe operation. The designer may also refer to the U.S. National Fire Protection Association (NFPA) - *NFPA 820 - Standard for Fire Protection in Wastewater Treatment and Collection Facilities.*

CHAPTER 4

ODOUR CONTROL

CHAPTER 4

ODOUR CONTROL

Sewage contains numerous potentially odourous substances, but the predominant groups are the reduced sulphur compounds. Of these, hydrogen sulphide (H_2S) is perhaps the most common and easily identified.

There are several texts which discuss odour generation, particularly as related to sulphides. The designer is referred to the U.S. EPA Design Manual, *Odour and Corrosion Control in Sanitary Sewage Systems and Treatment Plants* (EPA/625/1-85/018) and the ASCE Manual of Practice No. 69 *Sulfide in Wastewater Collection and Treatment Systems*.

4.1 ODOUR MEASUREMENTS AND LIMITS

Odour measurement is largely subjective. The most commonly accepted method of characterization is the Odour Unit (OU). The OU is based on the number of dilutions with clean air required to reach a threshold detection level. OU values are presented as an odour sample's Effective Dose - 50th percentile (ED50), meaning the number of dilutions at which an odour is detected by half the members of an odour panel using a dynamic dilution olfactometer. Thus, a sample which requires 4 dilutions to reach ED50 will contain 5 OU (4 dilutions plus the original volume).

The designer should determine, in consultation with the owner (and often the public), the appropriate odour limits and how they should be applied to the *sewage works*. Often a fence line odour limit is applied which determines the magnitude of the odours acceptable at the boundary of the facility. The limit will depend on the proximity of residential and commercial development and other site specific factors such as the proximity of parks, trails or roads and the sensitivity of the odour problem.

4.2 POTENTIAL ODOUR SOURCES

Odour problems tend to develop when dissolved hydrogen sulphide concentrations exceed 0.5 mg/L, or less if the pH is depressed. Production of sulphides commences in the collection system and will continue to occur wherever deposits accumulate under anaerobic conditions. The rate of sulphides production and odour generation are both temperature dependent. Industrial discharges frequently exacerbate odour. Typically, high sulphides content in discharges is most prevalent at low pH or high temperature conditions.

Turbulence promotes sulphides stripping and hence odours. In the collection system this occurs at drop manholes, sharp bends, forcemain discharge points and any hydraulic structure where turbulence or super-critical flow develops. Generally, the odour-emission potential at treatment plants decreases at each successive treatment stage. The influent sewer and headworks have a higher potential for odour emissions since they receive raw sewage with a higher sulphide content and are often turbulent areas. Preliminary treatment processes can generate odours from the screenings and grit handling areas. Aerated grit tanks will also strip sulphides.

Further sulphides generation often occurs as a result of anaerobic conditions in the sludge blankets accumulating in primary sedimentation tanks. The resultant hydrogen sulphide can be stripped at the effluent weirs due to the turbulence developed there. Aeration basins do not usually generate high sulphides odour, unless overloaded, as sulphides are oxidized within the basin. Final clarifiers rarely produce significant odours unless there are problems with the sludge or scum handling systems.

Solids handling and treatment processes have significant odour generation potential because of the high concentrations of sulphides present in sludge, scum and septage. Aerobic digesters, thickening and dewatering processes and sludge storage lagoons are all potential odour sources.

4.3 EVALUATION OF ODOUR PRODUCTION POTENTIAL

4.3.1 Monitoring Protocols

Detailed monitoring should be preceded by a preliminary study to analyze available data and odour complaints. Complaints should be correlated with data on plant operations, sewage characteristics and meteorological data. The preliminary study may include limited on-site sampling and analyses. Detailed field monitoring programs should be of sufficient duration to monitor seasonal variations in sulphides generation and hydrogen sulphide emissions. Monitoring should also include an hourly sampling and testing regime to identify typical diurnal fluctuations. Sampling points should be readily identifiable and remain consistent throughout the monitoring program.

4.3.2 Liquid Phase Analyses

Routine parameters to be monitored should include total and dissolved sulphides, BOD_5 or COD, temperature, pH and dissolved oxygen (DO). Oxidation-reduction potential (ORP) can also yield useful data. Additional analyses for TSS and particle size distribution will be needed if the results are to be used for predictive modelling of sulphides generation.

4.3.2.1 Gas Phase Analyses

In situ gas phase testing can be used to identify a wide range of odour producing compounds, including hydrogen sulphide, mercaptans and dimethyl disulphide. Continuous monitoring may be necessary in some cases to identify the peak hydrogen sulphide gas concentrations which trigger odour complaints. The designer should ensure that the equipment to be used for gas phase testing is suitable for the range of concentrations expected.

4.3.2.2 Air Sampling

Foul air sampling will be required if the intention is to use a dynamic dilution olfactometer and odour panel to determine OU values. Specialized sampling equipment and sample bags will be required.

4.3.2.3 Gas Chromatography

Analysis by gas chromatography (GC) is useful for identifying total levels of sulphides and other potential odour producing compounds. Analyses can be carried out on liquid or gas phase samples.

4.3.2.4 Interpretation of Results

Data obtained from monitoring programs provide the designer with useful information concerning the conditions governing sulphides generation and hydrogen sulphide release.

Areas of anaerobic activity producing sulphides are characterized by low dissolved oxygen or DO (less than 0.5 mg/L) and negative ORP. Reaction kinetics are temperature dependent. The rate of sulphides generation is greater in the presence of higher fractions of soluble BOD. Hydrogen sulphide emissions are increased at lower pH and higher temperature conditions.

4.3.3 Predictive Modelling

4.3.3.1 Sulphides Generation in Sewers and Forcemains

A number of predictive models have been developed for this purpose. The models have been developed from empirical data and are generally only valid for specific conditions. The designer is cautioned to ensure that the chosen method of analysis is applicable under the conditions in question.

4.3.3.2 Air Quality Computer Modelling

If designing to specific odour limits at identified receptor points, the designer should consider the use of a computer-based atmospheric dispersion model to simulate the behaviour of the odour plume.

4.4 ODOUR CONTROL AND ABATEMENT MEASURES

It is generally more reliable and cost effective to treat and remove odour producing compounds in the liquid phase rather than collecting and treating foul air. Sulphides develop in the collection system and the designer should therefore consider upstream control measures in conjunction with measures at the sewage treatment plant. Such measures should include designing to prevent deposition in sewers, minimizing residence time in pump station sumps and avoiding the use of siphons and long forcemains.

4.4.1 Prevention of Sulphides Formation

For new sewage treatment works, the designer should attempt to eliminate ‘dead zones’ where solids may accumulate. This can include increasing atmospheric turbulence (i.e., air entrainment) and attention to collection system design. An increase in atmospheric turbulence can be produced by several mechanical means, including adding structures and/or vegetation. Vegetation can increase local air turbulence and act as a filter. Design of gravity interceptors, tunnels, forcemains, siphons, wet wells and related facilities needs to include features to minimize the generation of sulfides and other odourous compounds formed by anaerobic biological activity. The design of the collection system will have an effect on the production and release of odours. Factors to be considered are as follows:

- Pipe slope;
- Transition structures;
- Manholes;
- Proximity to receptors; and
- Inverted siphons and forcemains.

Fillets should be incorporated into rectangular channels, conduits and tanks. Inverted siphons should be avoided. Aeration should be provided to channels and conduits where self-cleaning velocities cannot be achieved over the full flow range. Excessive aeration should be avoided because of potential odour generation due to increased turbulence.

Provide sufficient energy input per unit volume to ensure solids are maintained in suspension. The designer should also consider improved access provisions to facilitate routine housekeeping and cleaning activities.

4.4.2 Chemical Treatment

When dosing chemicals into sewage, side reactions will occur in addition to the desired reaction. In calculating dosing rates, the designer should allow a generous factor of safety to account for these side reactions. Pilot-testing should be considered for chemical dosing systems to establish appropriate dosages.

4.4.2.1 Oxidizing Agents

Chlorine (in gaseous form or as sodium hypochlorite solution), potassium permanganate and hydrogen peroxide will oxidize sulphides and inhibit sulphide production. Pure oxygen and air injection have also been used to raise DO levels in sewage.

4.4.2.2 Precipitants

Iron and zinc salts will precipitate sulphides. Ferrous and ferric chloride have been used in collection systems, forcemains and at sewage treatment plants.

The designer should consider the effect on the solids handling streams at the sewage treatment plant in terms of increased sludge production, increased levels of contaminants in the sludge and any corrosion implications.

4.4.2.3 pH Control

Intermittent slug dosing with sodium hydroxide will raise the pH, inhibiting sulphide production and preventing hydrogen sulphide off-gassing. This system is effective only in localized areas and should be considered only for specific problem areas in the collection system.

4.4.2.4 Electron Acceptors

Electron acceptors are taken up preferentially to the sulphate ion and thus prevent sulphide formation. Sodium nitrate has been used in lagoons for this purpose. Proprietary nitrate products have also been used in sewers.

4.4.2.5 Anthraquinone

Anthraquinone is a chemical that inhibits bacteria from using sulfate in their metabolic processes. It is only slightly soluble and should settle into the slime layer to become effective. When contacted by anthraquinone, the bacteria in the slime layer are inactivated for a period of several days up to six weeks. After this time, the bacteria start sulphide production again if not re-treated. Because of the low solubility, it is only partially effective in forcemain applications and for fast gravity main flows.

4.4.2.6 Caustic Slug Dosing

Sodium hydroxide is a strong caustic solution. It controls hydrogen sulphide by shifting the sulphide equilibrium from the H_2S form to the dissolved hydrosulfide (HS^-) form. The continuous addition of sodium hydroxide would prevent the release of H_2S, but is not a cost-effective solution. Periodic slug dosing with sodium hydroxide, however, can be effective in a sewer system. It works not by shifting the chemical equilibrium, but by inactivating the biological slime layer which is responsible for the generation of H_2S. The slime layer will regrow, but it will take several days or weeks for it to resume full sulphide production.

4.4.2.7 Nitrate Addition

Facultative and obligate anaerobic bacteria, which are responsible for sulphide production, prefer nitrate to sulphate as an electron acceptor. This results in the production of nitrogen gas and other nitrogenous compounds rather than hydrogen sulphide. Nitrate can be obtained in a variety of liquid and dry forms, mostly as sodium or calcium nitrate. It has several advantages over other control options, since nitrate:

- Is consumed more slowly than dissolved oxygen in sewer systems;

- Is nonflammable and nonhazardous, requiring no special containment or safety provisions; and
- Produces only minor flocculants and lower solids production.

4.4.2.8 Reaeration or Oxidation

The addition of oxygen to the sewage works can decrease odours from sewage since most odours are produced under anaerobic conditions within the sewage works. The addition of oxygen can directly oxidize the odour-causing compounds or create aerobic conditions necessary for aerobic bacteria to carry out this conversion. Through metabolic processes, aerobic bacteria prevent the formation of odourous compounds by outcompeting anaerobic bacteria for available substrate in the sewage.

The addition of pure oxygen gas accomplishes the same thing as the addition of air, but only approximately one-fifth as much volume is added to achieve the same dissolved oxygen concentration. Oxygen can either be generated on-site or purchased commercially. It has the further advantage of not containing nitrogen and thus it significantly reduces the potential for *air binding*. It also allows treatment of forcemains with longer detention times.

Air is a readily available source of oxygen. Air injection may cause turbulence since it comprises approximately four-fifths gasses other than oxygen, which will result in the release of odourous gasses. It has proven to be successful when injected at the head of short to moderate length forcemains. Problems have been encountered in forcemains that have high points since there is the potential for air binding and reduced flow capacity.

Ozone is an extremely powerful oxidant that can oxidize H_2S to elemental sulfur. Ozone is unstable and should be generated on-site. Ozone is a disinfectant. It is also toxic to humans at concentrations above 1 mg/L. Although it has been shown to reduce odours in air, effectiveness in reducing odourous compounds in sewage has not been documented. Since it is generated from air (although it can also be generated from pure oxygen), the problems associated with air injection into sewage also apply to ozone injection. Ozonation requires fairly sophisticated equipment, which is not readily utilized at unstaffed sites.

4.4.3 Control of Mass Transfer

The transfer of sulphides from liquid to gas phase can be reduced by minimizing liquid turbulence and reducing aeration. The designer should consider the following measures to reduce turbulence:

- Minimize elevation differences where streams converge;
- Introduce side streams below the liquid surface;
- Use submerged effluent weirs and downstream flow control in lieu of conventional launders for sedimentation tanks and clarifiers;

- Avoid excessive or unnecessary aeration; and
- Avoid the use of screw lift pumps on potentially odourous streams.

4.4.4 Foul Air Collection and Treatment

The following emission control systems should be considered for all solids handling areas and processes and for other areas of the facility where preventative measures are insufficient to mitigate odours.

Covers

Cover systems should be designed to minimize the number of joints. Seals should be provided at all joints. The designer should consider the corrosive action of sulphides and sulphuric acid when selecting cover materials and concrete coatings. Overhangs, ledges or lips on the underside of covers where condensate may collect should be avoided. The design of covers should be aimed at minimizing the volume of air requiring treatment.

The designer should consider operational requirements for access and cover removal and be aware that the installation of covers will create a confined space environment.

Ventilation

Ventilation rates should be based on the more stringent of two requirements: (a) maintain a slight negative pressure in the headspace and thus prevent fugitive odours escaping through joints in the cover system, and (b) limit sulphide concentrations in the air stream to a level that the downstream treatment systems can effectively treat.

Treatment Systems

There are numerous alternative treatment systems available. Selection of the appropriate system should be subject to a life-cycle cost-benefit analysis. The systems that may be considered include:

- Chemical scrubbers, packed bed or mist contactor types;
- Activated carbon, with or without chemical impregnation;
- Activated alumina impregnated with potassium permanganate;
- Biofilter, in-vessel or soil/compost types;
- Incineration (i.e., thermal oxidation); and
- Dual-stage systems comprising one or more of the above.

Design requirements for these systems vary considerably. The designer should consult equipment manufacturers for details.

When assessing bulk chemical storage requirements, the maximum effective storage life of the chemical should be considered. Many chemicals are temperature sensitive and storage tanks will require special provisions if located outdoors.

Provision will also be required for disposal of waste and side-streams from the treatment processes which may require further treatment before return to the main liquid train at the sewage treatment plant. An alternative form of biological treatment or pre-treatment involves blowing foul air through aeration tanks. Consideration and care should be given to potential blower corrosion and air-side diffuser fouling. For all sources of contaminants emission to the air within sewage works, the requirements of section 9 of the EPA need to be satisfied (*Section 3.11 - Emissions of Contaminants to Air*).

Operation and Maintenance

Reduction of the sludge storage on the treatment plant site should be considered to prevent odours if the sludge is susceptible to pH decay, in particular with unstabilized sludge. Adequate facilities should be provided for ventilation of any sludge storage or dewatering areas, if applicable. The exhaust air should be properly conditioned to avoid odour nuisance.

There are a number of operational procedures that can be utilized to limit the production or release of odours, the most important being good housekeeping. Routine hosing and debris removal at pump station wet wells and within the treatment plant can significantly reduce odour production. Operation of wet wells is also an important factor. While it may be more energy efficient to operate at higher wet well levels, this increases detention times and the potential for development of anaerobic conditions and H_2S production. Fill-and-draw pump stations should consider more frequent pumping while level set-points on variable speed pump stations should be lowered where odour is an issue.

The first step in any foul air treatment system is containment of the odourous air. If fugitive emissions under normal operation are not eliminated, the entire odour control strategy is negated. This applies to covered process tanks and channels and to occupied spaces. Collection of foul air from covered tanks and channels has traditionally been based on air exchange rates. A moderate exchange rate may be required to reduce condensation and corrosion. A higher exchange rate may be needed to allow utilization of the enclosed space above a clarifier or CSO tank.

Collection of foul air for prevention of air escape through cracks, leaks and other penetrations in a cover primarily depends on establishing a negative pressure within the enclosed headspace. The negative pressure is established by exhausting air from the enclosed headspace, which draws air into the headspace through the various openings in the cover. The negative pressure is a function of the air velocity through these openings. Factors to be considered in type and location of covers are:

- Permanency (fixed, removable);
- Ease of removal (by crane, manually);

- Accessibility/visibility (hatches, clear panels);
- Aesthetics (sun reflection, camouflage); and
- Sealing (gasketed, permanently sealed).

Odour containment is only effective if it is not compromised by leaving hatches or doors open or otherwise compromising the containment. It requires an ongoing education program to ensure that odour control procedures and design intentions are maintained.

While containment will increase the difficulty associated with operating covered units, it is important that every effort be made to minimize the inconvenience and maximize worker safety. For example, hatches which need to be opened to observe internal equipment should be readily accessible and easily opened.

4.5 SEPARATION DISTANCES BETWEEN SEWAGE WORKS AND SENSITIVE LAND USE

The separation distances specified in *ministry* Guideline D-2, *Compatibility between Sewage Treatment and Sensitive Land Use* are intended to mitigate the effects of offensive odours which may occur during normal operations or when facilities have minor overloads or upsets created by abnormal conditions or sludge processing/handling. Compliance with Guideline D-2, by providing adequate separation distances to mitigate odours, may also provide for adequate attenuation for noise associated with plant operation.

Separation distances should be measured from the periphery of the facility structure(s) that produce odours to the property lot-line of the sensitive land use and take into account any approved expansions to either the facility or the sensitive land use. Otherwise, it should be recognized that future expansions may be limited.

When new facilities or expansion to existing facilities are proposed, an adequate buffer area should be acquired as part of the project. Where an adequate buffer area cannot be purchased or provided, more effective technical mitigation procedures would be necessary to provide an optimum level of protection between the sewage treatment facility and affected sensitive land uses. The designer should consider covering certain sections of the facility and treating collected off-gases. In some cases a combination of distance, covering and treatment of collected off-gases may be required.

Recommended separation distances for sewage treatment plants (STPs) in proportion to their rated capacities are provided in subsequent sections.

4.5.1 STPs with Rated Capacity Equal to or Less than 500 m^3/d (0.13 mUSgd)

For STPs with a rated *capacity* of less than 500 m^3/d (0.13 mUSgd), the recommended separation distance is 100 m (330 ft). A separation distance of less than 100 m (330 ft) may be acceptable, however a qualified professional would be required to produce a study demonstrating the suitability of the distance based on:

- The degree and type of odour mitigation applied to the facility;
- Other contaminants of concern (i.e., aerosols) which may need to be addressed; and
- The application of noise reduction equipment to any potential noise source(s).

4.5.2 STPs with Rated Capacity Greater than 500 m^3/d (0.13 mUSgd) but Less than 25,000 m^3/d (6.6 mUSgd)

The recommended separation distance is 150 m (490 ft) with the minimum separation distance of not less than 100 m (330 ft).

4.5.3 STPs with Rated Capacity Equal to or Greater than 25,000 m^3/d (6.6 mUSgd)

The recommended separation distance should be greater than 150 m (490 ft) and determined on a case-by-case basis. The determination of the required distance will depend on factors such as the type of treatment process, type of noise or odour control measures being applied, existing municipal zoning and availability of land.

4.5.4 Sewage Lagoons

Notwithstanding sewage treatment plant requirements, the recommended separation distance for lagoons varies from 100 to 400 m (330 to 1,300 ft). A suitable separation distance should be determined by a qualified consultant on a case-by-case basis. In determining a suitable separation distance, considerations may include factors such as lagoon design, hauled sewage or sludge handling and whether there is a supplemental aeration system. Odours and the aerosols emitted from the lagoons are also dependant upon the operation, design, environmental conditions and the season in some cases.

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CHAPTER 5

DESIGN OF SEWERS

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DESIGN OF SEWERS

This chapter describes design guidelines associated with the collection and transport of sewage and stormwater. The items described in this chapter include storm sewers and sanitary sewers, as well as alternative sewer types, sewer system rehabilitation, manholes, pipe design and inverted siphons. A section is included on sewers with streams and aerial crossings and protection of drinking water supplies.

5.1 INTRODUCTION

These design guidelines are intended to assist consulting and municipal engineers and other designers in the preparation of sewer system designs. Municipalities, in which the *sewage works* will be constructed, may have servicing standards that exceed the requirements of these guidelines. The designer should ensure that they are aware of the requirements of all other approving authorities prior to finalizing any designs.

Although some aspects of the guidelines relate only to municipal services, the guidelines are meant to apply to other sewer systems serving developments such as mobile home parks and condominiums which also require *ministry* approval under the *Ontario Water Resources Act* (OWRA) (*Section 1.5 - Ministry Approval Program for Sewage Works*).

5.2 SEPARATE VERSUS COMBINED SEWERS

All new sewer construction in Ontario should be of the separate type, with stormwater and *groundwater* flow being excluded to the maximum possible extent. Stormwater and groundwater flow consist of rain, water from roofs, streets and other areas and groundwater from foundation drains.

A combined sewer system is a collection system designed to convey both sanitary sewage and stormwater runoff through a single-pipe system to a sewage treatment works. See *Section 8.3.4 Combined Sewer System* for additional related information.

New combined sewer systems will not be approved by the ministry. New storm drainage systems will not be permitted to connect to existing combined sewers except as an interim measure where circumstances allow no other alternative. In such cases, the proponents will be required to provide the justification for the continued use of combined sewers along with a plan and a timetable for the ultimate disposition of the storm drainage. The designer should refer to *Chapter 21 - Control and Treatment of Combined Sewer Overflows* for information on combined sewer overflow treatment needs.

Since the ministry discourages the extension of combined sewers, the design of such systems is excluded from this document.

5.3 STORMWATER MANAGEMENT

Stormwater management is required to mitigate the effects of urbanization on the hydrologic cycle including increased runoff and decreased infiltration of rain and snowmelt. The ministry's *Stormwater Management Planning and Design Manual* (2003) provides guidance for planning and designing stormwater management systems to meet the multiple objectives of stormwater management: maintaining the hydrologic cycle, protection of water quality and preventing increased erosion and flooding.

The designer should refer to the *Stormwater Management Planning and Design Manual* (2003) for design guidance for individual lot level, conveyance, and end-of-pipe practices. The manual includes physical constraints such as, soil type and depth to groundwater, as well as sizing and configuration and design details including inlets and outlets, filter media and distribution pipes. The guidance also includes cold climate considerations and the incorporation of vegetation in system design.

For additional information the designer should refer to *Section 21.3 - Source Management and Control Technologies.*

5.4 STORM SEWERS

The following subsections cover flow calculations pertaining to the design of separate storm sewer systems.

5.4.1 Runoff Computation

The peak rate of runoff from an area may be calculated using the Rational Method, using the following formula:

$$Q = 2.78\, C \cdot i \cdot A$$

where

Q = Peak flow (L/s)
A = Area in hectares
i = Average rainfall intensity in millimetres per hour for a duration equal to the time of concentration for a particular storm frequency. The time of concentration is the time required for stormwater runoff to flow from the most remote point of a *watershed* or drainage area to the outlet or point under consideration.
C = Runoff coefficient; see Table 5-1 for typical values

This formula is used for sizing storm sewers to remove water as fast as possible from street surfaces for the specific design storm frequency. When the calculation of the volume of runoff or the rate over time (hydrograph) is needed, then a hydrologic simulation model will be necessary.

Calculations based on a hydrologic simulation model are preferable. For systems serving large areas, or involving treatment and/or storage systems, the use of a model may be necessary.

The remainder of this section will deal only with the Rational Method. Reference should also be made to the Ministry of Transportation and Communication (MTC) *Drainage Manual* (1980), for a further discussion of the Rational Method for storm sewer design. This MTC Manual also covers culvert and open channel design guidelines.

5.4.2 Drainage Area

The drainage area to be used in the design of a storm sewer system should include all those areas which will reasonably or naturally drain to the system.

The area term in the Rational Method formula represents the total area tributary to the point on the storm sewer system under consideration.

5.4.3 Rainfall Intensity

The rainfall intensity for a specific storm frequency and time of concentration should be determined from intensity duration - frequency curves applicable for the municipality in which the system is to be constructed.

For a discussion of rainfall intensity curves, reference should be made to The Urban Drainage Subcommittee of the Canada-Ontario Agreement on Great Lakes Water Quality *Manual of Practice on Urban Drainage* (1987) and Ministry of the Environment and the Municipal Engineers Association *Municipal Works Design Manual* (1984).

5.4.4 Design Storm Frequency and Runoff Coefficient

The storm frequency in the design of stormwater conveyance systems will vary depending upon the nature of the area being served, the value of the property being protected and the consequences of more intense storms being experienced.

It is recommended that the major-minor drainage system approach be utilized for urban drainage for all future development. The minor drainage system (i.e., roof gutters, service connections, street gutters, catch basins and storm sewers) accommodates the runoff from more frequent storms up to the design frequency of the system (e.g. 2-year return design storm). Where weepers/foundation drains are connected to the storm sewers, they should also be designed to capture no more than the amount of runoff from design frequency storm in order to prevent surcharge conditions.

The major system (i.e., natural streams and valleys and the roads, swales, channels and ponds) accommodates runoff from less frequent design storms such as the 100-year return design storm or regional flood event.

Although the designer should consult the local weather data to establish the design storm frequency criteria for the design of stormwater conveyance systems, the level of convenience provided by the minor system is a decision of the municipality, or the owner of the system in the case of private systems. It is recommended that as a minimum a 2-year return design storm should be used for design purposes. For information on design details of major-minor systems the designer should refer to the ministry document *Stormwater Management Planning and Design Manual* (2003).

The following ranges of runoff coefficients shown in Table 5.1 are considered reasonable for design purposes:

Table 5-1 - Runoff Coefficients

Sources	**Coefficient (C)**
Asphalt, concrete, roof areas	0.90-1.00
Grassed areas, parkland	0.15-0.35
Commercial	0.75-0.85
Industrial	0.65-0.75
Residential:	
Single Family	0.40-0.45
Semi-detached	0.45-0.60
Row housing, Townhouses	0.50-0.70
Apartments	0.60-0.75
Institutional	0.40-0.75

The runoff coefficient for any particular type of area should the upper value of the above tabulated ranges to account for antecedent precipitation conditions when expected runoff is being calculated for less frequent, high intensity storms. The lower value of the range may be used for shorter recurrence interval storms under conditions of moderate to flat slopes and permeable soils.

The time of concentration is the time required for flow to reach a particular point in the sewer system from the most remote part of the drainage area. It includes not only the travel time in the sewers, but also the inlet time or time required to flow overland into the sewer system.

Inlet times should be calculated rather than relying upon arbitrary minimum or maximum times. The calculation should be based upon the overland flow route that will exist when the sewer system has been fully developed to the drainage limit. In the case of single-family residential areas, calculations will not be needed if a maximum inlet time of 10 minutes is used. Sewers should be designed with the ability to overflow to avoid emergency and/or unavoidable conditions. For details on such conditions refer to *Section 8.3 - Definition of Terms.*

5.4.5 Catch Basins

Catch basins may be installed with or without sumps. The minimum pipe size for a single catch basin lead should be 200 mm (nominal pipe size of 8 in; NPS-8) in diameter and 250 mm (NPS-10) for a double catch basin.

Catch basins should be provided at adequate intervals in the sewer system to ensure that the road drainage is able to be intercepted up to the capacity of the storm sewer. The spacing will vary with the road width, grade and crossfall and with the design storm frequency. The spacing will also be affected by the location of pedestrian crossing points, intersections, low points and driveway depressions. In general, for pavement widths up to 9.8 m (32 ft) with two per cent crossfall, the maximum spacing that should be used is shown in Table 5.2.

Table 5-2 - Recommended Spacing for Catch Basins

Road Gradient (%)	Maximum Spacing
0 to 3	110 m (350 ft)
3.1 to 4.5	90 m (300 ft)
Over 4.5	75 m (250 ft)

Stormwater management systems using inlet control catch basins may be designed with less frequent spacing than those outlined above. In such cases, the designers should justify whatever spacing is used.

5.4.6 Storm Sewer Gratings

The inlets and outlets of piped sections of stormwater management systems which are accessible to the public should be provided with protective devices. As a minimum, it is recommended that inlets and outlets of pipes 600 mm diameter (NPS-24), or larger, should be provided with gratings to prevent small children from gaining access to the sewers. Grating bars should be spaced 150 mm (6 in) apart. For large inlet structures, inclined gratings may be necessary to prevent water pressure from trapping a person against the grating. Such inclines will also tend to make the gratings self-cleaning from a debris standpoint.

5.4.7 Foundation Drainage

It is recommended that foundation drainage be directed either to the surface of the ground or storm sewer system, if one exists.

The designer should consider and advise the municipality of the following factors:

- Possibility of storm sewer surcharging;
- Difference in elevation between basement floor slabs and storm sewer obverts;

- Possibility of foundation damage and flooding which could result due to back up into private storm drains;
- Where concerns exist regarding the first two points, but where connection to a storm sewer is still desirable, this connection should be made via a sump pump system; and
- The use of a "third" pipe or foundation drain collector.

Where foundation drains are to be connected by gravity to the storm sewers, the designer should consider the following options in order to reduce the probability/frequency of foundation drain surcharging:

- The use of a higher return frequency in the design of the storm sewer;
- The construction of a deeper sewer with the depth of the sewer being determined/checked by the hydraulic grade line for surcharged conditions; and
- Inlet controls or increased spacing of the inlets to prevent water from gaining access to sewers at a rate greater than the design storm event.

The designer should recognize that the state-of-the-art of inlet control design has not reached the point where performance can be guaranteed.

Foundation drains for industrial, commercial and institutional buildings should not be directed to sanitary sewers.

The connection of the foundation drains to a sanitary sewer system is strongly discouraged by the ministry because of the serious negative impact such connections might have on the system and the operation of the sewage treatment plant.

5.4.8 Storm Sewer Design

It is recommended that storm sewer capacities be calculated using the Manning formula with a roughness coefficient (n) of 0.013 for all smooth-walled pipe materials (*Section 5.7.1 - Flow formulas and Roughness Coefficient)*. With corrugated metal pipe, the roughness coefficient "n" should be 0.024 for plain pipe, 0.020 for paved invert pipe and 0.013 for fully paved pipe.

For a more complete discussion on manhole spacing, hydraulics, pipe design, depth of cover and pipe anchoring the designer should refer to *Section 5.7 - Details of Design and Construction of Sewers*.

5.5 SANITARY SEWERS

5.5.1 Design Period and Tributary Area

Wherever possible, the design of sanitary sewers should be based on the ultimate sewage flows expected from the tributary area. Tributary areas need not necessarily be restricted to current municipal limits. In cases where the tributary area is poorly defined, or where the financial burden on present users would be too severe, the sewage system design may be based on more restricted approaches. In these cases the design period should be at least 20 years.

For the estimation of future sewage flow rates for municipal sewage collection systems, the designer should make reference to the Official Plan (or Draft Official Plan) of the municipality. Such official plans will contain future population densities and land uses.

If no official plan or draft plan exist, the designer should size sanitary sewers for population densities of at least 25 persons per gross hectare. This minimum level of population density will generally be suitable for rural municipalities only. If the municipality already has higher population densities, the designer should use similar or higher densities for new growth areas.

Sanitary sewer capacities should be designed for the estimated ultimate tributary population, except where parts of the systems can be readily increased in capacity. Similarly, consideration should be given to the maximum anticipated capacity of institutions, industrial parks and other sewage sources. For details on design flows refer to *Section 8.5.4 - Sewage flows*.

5.5.2 Design Sewage Flows

Sanitary sewage flows are made up of wastewater discharges from residential, commercial, institutional and industrial establishments, plus extraneous flow components from such sources as groundwater and surface runoff.

The peak sewage flow rates, for which sewer system capacity is to be provided, should be calculated for all flow contributors, for present and future conditions. In addition to being able to carry the peak flows, sewers should be able to develop sufficient flow velocity to transport the sewage solids, thus avoiding deposition and the development of nuisance conditions under lesser flow rates.

5.5.2.1 Domestic Sewage Flows

The following criteria should be used in determining peak sewage flows for municipal sewer design for residential areas:

- Design population derived from drainage area and expected maximum population over the design period;
- Average daily domestic flow (exclusive of extraneous flows) of 225 to 450 L/(cap·d) [59 to 119 US gal/(cap·d)];
- Peak Extraneous Flow; and
- Peak domestic sewage flows to be calculated using the following formula:

$$Q\,(d) = \frac{PqM}{86.4} + IA$$

where

Q (d)	=	Peak domestic sewage flow (including extraneous flows) in L/s
P	=	Design population, in thousands
q	=	Average daily per capita domestic flow in L/cap·d (exclusive of extraneous flows)
I	=	Unit of peak extraneous flow, in L/(ha·s); applicable references should be consulted for values
A	=	Gross tributary area in hectares
M	=	Peaking factor (as determined from Harmon or Babbitt Formula)

Harmon Formula

$$M = 1 + \frac{14}{4 + P^{0..5}}$$

Babbitt Formula

$$M = \frac{5}{P^{0.2}}$$

Note that the minimum permissible peaking factor M is 2.0.

5.5.2.2 Commercial and Institutional Sewage Flows

The sewage flows from commercial and institutional establishments vary greatly with the type of water-using facilities present in the development, the population at the facilities, the presence of water metering and the extent of extraneous flows entering the sewers.

Institutional flows should be computed for each individual case based on historical records, when available. Where no records are available, the unit values in Table 5-3 should be used. For commercial and tourist areas, a minimum allowance of 28 m^3/(ha·d) [2,993 US gal/(ac·d)] average flow should be used in the absence of reliable flow data.

For individual commercial and institutional uses the sewage flow rates shown in Table 5-3 are commonly used for design.

The peaking factors applicable for sewage flows from individual establishments will be similar to the relative peak water usage rates. The designer should refer to the ministry *Design Guidelines for Drinking Water Systems* for details.

Table 5-3 - Common Sewage Flow Rates for Commercial and Institutional Uses

Use	Unit Sewage Flow[1]		Flow Unit per
	SI (L/d)	US (US gal/d)	
Shopping Centre	2.5-5.0	0.7-1.3	based on total floor area (m^2 and ft^2)
Hospitals	900-1800	237-475	bed
Schools	70-140	18.5-37	student
Travel Trailer Parks	340 800	90 211	space (min. without water hook-ups) space (min. with indiv. water hook-ups)
Campgrounds	225-570	59-150	campsite
Mobile Home Parks	1000	264	parking space
Motels	150-200	40-53	bed space
Hotels	225	59	bed space

Note:
1. Unit sewage flow rates exclusive of extraneous flows.

5.5.2.3 Industrial Sewage Flows

Peak sewage flow rates from industrial areas vary greatly with the extent of area development, the types of industries present, the provision of in-plant effluent treatment and recycle/re-use or rate of flow controls, the presence of cooling waters in the discharge and other factors. Due to the occasional presence of individual industrial water supplies, the rates of water supply from municipal systems into industrial areas will not always be indicative of the sanitary sewage flows to be expected. The discharge of cooling water from municipal supplies into storm sewers or *surface water* courses may result in lower flows in sanitary sewers than would be expected based on municipal water usage.

The calculation of design average and peak sewage flow rates for industrial areas is industry/process specific and may be difficult to predict accurately. Improving flow prediction may include better monitoring of industries present in industrial areas. In this way, a reasonable allowance can be made for peak industrial sewage flows for an area and by flow monitoring allowances can be better maintained. Industries with the potential to discharge sewage at higher than the accepted rates may be required to provide flow equalization with discharge at off-peak discharge periods.

5.5.2.4 Foundation Drainage

It is essential that foundation drainage be directed to storm sewer systems or in accordance with local municipal best management practices taking into account on-site drainage/infiltration conditions. Since sanitary sewers are not designed to accept these flows (i.e., rainwater leaders and/or foundation drains), serious damage/problems may result, such as cracking of basement floor slabs or flooding of basements if foundation drainage is discharged to the sanitary sewers (*Section 5.4.7 - Foundation Drainage*).

5.5.2.5 Extraneous Sewage Flows

When designing a sanitary sewer system, an allowance should be made for the leakage of groundwater into the sewers and building sewer connections (infiltration) and for other extraneous water entering the sewers from sources such as manhole covers (inflow).

Due to the extremely high peak flows that can result from roof downspouts, they should not, under any circumstance, be connected directly or indirectly via foundation drains to sanitary sewers.

The amount of groundwater leakage into the sewer system will vary with the quality of construction, type of joints, ground conditions and level of groundwater in relation to pipe. Although such infiltration can be reduced by proper design, construction and maintenance, it cannot be completely eliminated and an allowance should be made in the design sewage flows to cover this flow component. Despite the fact that these allowances are generally referred to as infiltration allowances, they are intended to cover the peak extraneous flows from all sources (i.e., infiltration and inflow) likely to contribute non-waste flows to the sewer system.

5.6 ODOURS AND CORROSION IN SEWERS

Sewers are designed to accommodate future population growth and high flow events. As a result, during low flow periods, either early in the design life of the sewer or during dry weather periods (or both), sewage velocities are reduced and solids deposition and stagnation conditions can occur. Solids deposition and slime growth can lead to anaerobic conditions and the generation of inorganic gases including hydrogen sulphide (H_2S), ammonia and carbon dioxide. H_2S is the most common corrosive and odorous gas produced in sanitary sewers. Once released to the sewer atmosphere, anaerobic bacteria which reside in sewer wall slimes convert the H_2S to sulphurous and sulphuric acid which corrodes concrete sewer walls.

A number of factors affect H_2S production in a sewer system including: settleable solids, organic material, nutrients, sulphates, dissolved oxygen, temperature, flow turbulence, velocity, surface area and detention time.

The designer should evaluate areas or portions of the sewer that promote either gas generation or ventilation. Air is normally drawn down sanitary sewers by the flow of sewage. There is also an exchange of air in the headspace as a result of an increase or decrease in flow. As the flow rate increases, the headspace air is exhausted through openings in the system, such as manhole cover openings or grating. As the flow decreases, air is drawn into the headspace in the sewer. Inverted siphons also provide a potential for gas generation due to the stagnant volume of sewage that is maintained in these pipes. This can be managed with multiple barrels and self-cleaning velocities.

Designers should recognize the potential for anaerobic conditions and gas generation in sewers and design with self-cleansing velocities, as well as provision for regular maintenance and flushing. Dissolved sulphides can be controlled by liquid-phase treatment methods such as oxygenation, chemical oxidants, precipitation, pH stabilization and nitrate addition. Once gas is released to the headspace, it needs to be collected and treated using gas phase treatment methods.

The designer should refer to *Chapter 4 - Odour Control* for more information.

5.7 DETAILS OF DESIGN AND CONSTRUCTION OF SEWERS

5.7.1 Flow Formulas and Roughness Coefficient

It is recommended that sanitary sewers be designed using either the Chézy-Kutter or Chézy-Manning formula and a roughness coefficient (*n*) of no lower than 0.013 for all smooth-walled pipe materials.

The Chézy-Manning formula, which is the most commonly used formula for calculating sewer capacity, is as follows:

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2}$$

where

Q	=	Flow capacity of sewer (m^3/s)
R	=	Hydraulic radius of pipe (m)
S	=	Sewer slope (m/m)
n	=	Manning roughness coefficient (unitless)
A	=	Area (m^2)

For sewers with less than 900 mm diameter (NPS-36)), Chézy-Kutter's formula gives a more conservative estimate of sewer capacity. For this reason, the Chézy-Kutter's formula is usually used to calculate minimum acceptable sewer slopes (*5.7.5 - Slope*).

5.7.2 Minimum Size

No gravity sewer conveying raw sewage within a municipal sanitary sewer system should be less than 200 mm diameter (NPS-8). The minimum storm sewer diameter is 250 mm (NPS-10). Individual service connections as small as 100 mm diameter (NPS-4) may be considered by the designer.

5.7.3 Depth

Sewers should be sufficiently deep to receive sewage from basements and to prevent freezing and damage due to frost. Insulation should be provided for sewers that cannot be placed at a depth sufficient to prevent freezing.

For buildings substantially below street level, it may be more economical to pump into the sewer rather than deepen the sewer to accommodate a limited number of low-lying properties. To allow for gravity drainage from basements, sewer inverts should normally be at least 0.9 to 1.5 m (3 to 5 ft) below basement floor levels.

Reference should be made to *Chapter 6 - Challenging Conditions Affecting Servicing* for a recommended approach in calculating frost penetration depths.

Other factors which can affect sewer depth are interference with other utilities at crossings (both main sewer and building sewer vertical alignments can be affected by storm sewers, watermains and gas mains) and length of building sewer connections.

5.7.4 Buoyancy

Buoyancy of sewers should be considered and flotation of pipes should be prevented with appropriate construction where high groundwater conditions are anticipated.

5.7.5 Slope

All sewers should be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/s (2.0 ft/s), based on the Chézy-Manning formula using an *n* value of 0.013. The Kutter formula can also be used. Table 5-4 provides the recommended minimum slopes which should be provided for sewers 1050 mm in diameter (NPS-42) or less; however, slopes greater than these values may be desirable to control sewer gases or to maintain self-cleansing velocities at all rates of flow within the design limits. The Kutter or Chézy's roughness coefficient is used in the Chézy-Kutter and is related to the Manning roughness coefficient *n* by the formula below.

$$Q = A \cdot C\sqrt{R \cdot S}$$

and

$$C = \frac{k + \frac{k_2}{S} + \frac{k_3}{n}}{1 + \frac{n}{\sqrt{R}}\left(k_1 + \frac{k_2}{S}\right)}$$

where

A = pipe cross-sectional area (m^2 (ft^2))
C = Chézy's (or Kutter) roughness coefficient ($m^{1/2}/s$, ($ft^{1/2}/s$))
S = Friction slope (m/m, (ft/ft))

R = Hydraulic roughness (unitless)
N = Kutter's roughness (unitless)
k_1 = Constant (23.0 SI, 41.65 US)
k_2 = Constant (0.00155 SI, 0.00281 US)
k_3 = Constant (1.0 SI, 1.811 US)

Sewers 1200 mm in diameter (NPS-48) or larger should be designed and constructed to give mean velocities, when flowing full, of not less than 0.9 m/s (3.0 ft/s), based on the Chezy-Manning formula using an *n* value of 0.013.

5.7.5.1 Allowances for Hydraulic Losses at Sewer Manholes

The following minimum allowances should be made for hydraulic losses incurred at sewer manholes:

Manhole Type	**Loss Incurred**
Straight Run	Grade of Sewer
45º Turn	0.03 m (1.2 in)
90º Turn	0.06 m (2.4 in)
Junctions and Transitions	Physical modeling recommended

Although the needed invert drops for the above hydraulic losses will be adequate for sewers with flows at the low end of the acceptable velocity range, the required drops should be specifically calculated for high velocity sewers.

5.7.5.2 Minimum Flow Depths

Slopes less than those required for a 0.6 m/s (2.0 ft/s) velocity when flowing full may be considered when increasing the slope would require deepening of extensive sections of the system or the addition of a pumping station. In such instances, the reduction of the slope would only apply to 200 mm (NPS-8) and 250 mm (NPS-10) diameter pipes, with the minimum allowable slope being 0.28% for 200 mm (NPS-8) pipe and 0.22% for 250 mm (NPS-10) pipe. Such decreased slopes may be considered where the depth of flow will be 0.3 of the diameter or greater for design average daily flow.

5.7.5.3 Solids Deposition

The pipe diameter and slope should be selected to obtain the greatest practical velocities to minimize solids settling problems. Oversized sewers should not be used to justify using flatter slopes. If the proposed slope is less than the minimum slope of the smallest pipe which can accommodate the design peak hourly flow, the actual depths and velocities at minimum, average and design peak day and design peak hourly flow for each section of the sewer should be calculated by the designer.

Table 5-4 - Recommended Minimum Slopes for Various Sewer Sizes

Nominal Sewer Size	Minimum Slope in m /100 m (Feet per 100 Feet)[1]
200 mm (8 inch) (NPS-8)	0.40
250 mm (10 inch) (NPS-10)	0.28
300 mm (12 inch) (NPS-12)	0.22
350 mm (14 inch) (NPS-14)	0.17
375 mm (15 inch) (NPS-15)	0.15
400 mm (16 inch) (NPS-16)	0.14
450 mm (18 inch) (NPS-18)	0.12
525 mm (21 inch) (NPS-21)	0.10
600 mm (24 inch) (NPS-24)	0.08
675 mm (27 inch) (NPS-27)	0.067
750 mm (30 inch) (NPS-30)	0.058
825 mm (33 inch) (NPS-33)	0.052
900 mm (36 inch) (NPS-36)	0.046
975 mm (39 inch) (NPS-39)	0.041
1050 mm (42 inch) (NPS-42)	0.037

[1] These minimum slope recommendations should be consistent with local municipal design requirements.

5.7.6 Maximum and Minimum Velocities

All sewers should be designed with such slopes that they will have a minimum sewage flow velocity, when flowing full, of at least 0.6 m/s (2.0 ft/s). In cases where the flow depth in the sewer, under peak flow, will not be 0.3 of the pipe diameter or greater, the actual flow velocity at peak flow should be calculated using a hydraulic elements chart and the slope increased to achieve adequate flushing velocities. In certain circumstances, such as where increased slopes would require deepening of extensive sections of the sewage collection system or the addition of a pumping station, peak sewage flow velocities of less than 0.6 m/s (2.0 ft/s) may be considered provided that the municipality accepts that there may be increased maintenance requirements.

It should be noted that sewers achieving flow velocities less than those required for self-cleansing of grit and organics may have increased maintenance expenses and need frequent cleaning due to the deposition of solids. These increased maintenance costs should be compared with the costs which would have been incurred if sewers were deepened to achieve adequate slopes.

In sizing sanitary sewers and selecting sewer slopes, consideration should be given to possible sulphide generation problems. Sulphide problems can be minimized by designing for sewers to flow less than full under peak flow conditions and to flow at velocities of 0.6 m/s (2.0 ft/s) or higher.

The velocities in sanitary sewer systems should not be more than 3 m/s (10 ft/s), especially where high grit loads are expected. Higher velocities should be avoided unless special precautions are taken. Where velocities greater than 4.6 m/s (15 ft/s) are attained, special provision should be made to protect against pipe displacement by impact and erosion. Velocities in storm sewers should not be greater than 6 m/s (20 ft/s).

5.7.6.1 Steep Slope Protection

Sewers on 20 percent slopes or greater should be anchored securely with concrete anchors spaced as follows:

- Not over 11 m (36 ft) centre to centre on grades 20 percent and up to 35 percent;
- Not over 7.3 m (24 ft) centre to centre on grades 35 percent and up to 50 percent; and
- Not over 4.9 m (16 ft) centre to centre on grades 50 percent and over.

5.7.7 Alignment

In general, sewers equal to or less than 600 mm diameter (NPS-24) should be laid with straight alignment between manholes. Straight alignment should be checked by either using a laser beam or lamping.

Curvilinear alignment of sewers larger than 600 mm in diameter (NPS-24) may be considered on a case-by-case basis provided compression joints are specified and American Society for Testing and Materials (ASTM) or specific pipe manufacturers' maximum allowable pipe joint deflection limits are not exceeded. Curvilinear sewers should be limited to simple curves which start and end at manholes. When curvilinear sewers are proposed, the recommended minimum slopes indicated in Table 5-4 should be increased accordingly to provide a minimum velocity of 0.6 m/s (2.0 ft/s) when flowing full.

Sanitary sewers are generally located at or near the centreline of roads to allow buildings on both sides of the street to be serviced with approximately the same lengths of building sewers. Municipalities generally have standards on the preferred location of services. These standards should be compared with the ministry Guideline F-6, *Sewer and Watermain Installation: Separation Distance Requirements.*

5.7.8 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the 0.8 depth point of both sewers at the same elevation.

Sewer extensions should be designed for projected flows even when the diameter of the receiving sewer is less than the diameter of the proposed extension. The connection should be done at a manhole constructed with an appropriate flow channel to minimize turbulence.

5.7.9 Pipe Materials

The pipe material selected should be adapted to local conditions, such as:

- Character of industrial wastes;
- Possibility of septicity;
- Soil characteristics;
- Exceptionally heavy external loadings;
- Abrasion; and
- Corrosion.

Sewers should be designed to prevent damage from superimposed live, dead and frost induced loads. Proper allowance for loads on the sewer should be made for soil type, groundwater conditions, as well as the width and depth of the trench. Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction should be used to withstand potential superimposed loading or loss of trench wall stability. The designer should refer to the *Ontario Provincial Standards for Roads and Public Works* (OPS) (http://www.ops.on.ca) for details on sewer pipe materials and installation.

Suitable couplings complying with ASTM specifications should be used for joining dissimilar materials.

For new pipe materials, for which ASTM standards have not been established, the design engineer should provide complete pipe specifications and installation specifications developed on the basis of criteria adequately documented and certified in writing by the pipe manufacturer.

For sewer applications requiring pressure pipe, the designer should refer to *Section 5.14 Protection of Drinking Water Systems*.

In choosing pipe material, the designer should consider the following factors:

- Life expectancy and use experience;
- Resistance to scour;
- Resistance to acids, alkalis, gasses and solvents;

- Ease of handling and installation;
- Physical strength;
- Type of joint - water tightness and ease of assembly;
- Availability and ease of installation of fittings and connections;
- Availability in sizes required; and
- Cost of materials, handling and installation.

5.7.10 Installation

Installation specifications should contain appropriate requirements based on the criteria and standards established by industry in its technical publications. Requirements should be set forth in the specifications for the pipe and methods of bedding and backfilling thereof so as not to damage the pipe or its joints, impede cleaning operations and future tapping, nor create excessive side fill pressures and ovalation of the pipe, nor seriously impair flow capacity.

Excavation for placing sewer pipes, backfilling and compacting should be specified in accordance with *Ontario Provincial Standards Specifications* (OPSS) 514, *Construction Specifications for Trenching, Backfilling and Compacting*. Final backfill should be placed in such a manner as not to disturb the alignment of the pipe.

Ring deflection testing should be performed on all sewers constructed using plastic pipe. The designer should reference OPSS 410, *Construction Specifications for Pipe Sewer Installation in Open Cut* for details on the testing procedure.

5.7.11 Joints and Infiltration

5.7.11.1 Joints

The types of joints and the materials used should be included in the specifications. Sewer joints should be designed to minimize infiltration and to prevent the entrance of roots throughout the life of the system.

5.7.11.2 Service Connections

Service connections to the main sewer should be made using factory made tees or wyes, strap-on-saddles or other approved saddles. Factory made tees or wyes should be used for all service connections where the diameter of the main pipe sewer is:

- Less than 450 mm (NPS-18); or
- Less than twice the diameter of the service connection.

Strap-on-saddles should be installed before laying the pipe.

Holes in the main sewer pipe should be cut with approved cutters and should be the minimum diameter required to accept the service connection saddle. If mortar-on saddles are used, the inside of the pipe should be mortared at the connection. Service connections should be plugged at the property line with watertight caps or plugs. Plugs or caps should be braced sufficiently to withstand hydrostatic or air test pressures.

The designer of sanitary sewer service connections should consider:

- Minimum Diameter: For gravity flow, 100 mm (NPS-4) or pipe size needed to satisfy requirement of the Part 7 of Division B of the *Building Code* (Ontario Regulation 350/06) made under the *Building Code Act, 1992;*
- Sanitary Sewer Service Connection Grades;
  - Recommended Grade 2%; and
  - Minimum Grade 1%;
- Materials: Reference should be made to the Ontario Provincial Standards - OPSS 410 for acceptable alternate materials for services; and
- Risers: Service risers from main sewers buried more than 4.0 m (13 ft) should be taken off at an angle not less than 45° from the vertical, moved to the vertical by an appropriate elbow and the vertical section provided with a slide fitting. Alternatively, where the main sewer depth is greater than 4.0 m (13 ft), the use of a shallow "local" collector sewer could be considered with service connections made to the shallow sewer. The designer should consult the municipality for local design requirements.

5.7.11.3 Leakage Tests

Leakage tests should be specified. This may include appropriate water or low pressure air testing. The testing methods selected should take into consideration the range in groundwater elevations during the test and anticipated during the design life of the sewer.

5.7.11.4 Water (Hydrostatic) Test

The test section should be slowly filled with water making sure that all air is removed from the line. A period of 24 hours for absorption should be allowed before starting the test except if exfiltration requirements are met by a test carried out during the absorption period.

Water should be added to the pipeline prior to testing until there is a head in the upstream manhole hole of 600 mm (NPS-24) minimum over the crown of the pipe or at least 600 mm (24 in) above the existing groundwater level, whichever is greater. The maximum limit of the net internal head on the line is 8 m (26 ft). In calculating net internal head, allowance for groundwater head, if any, should be made.

The distance from the manhole frame to the surface of the water should be measured. After allowing the water to stand for one hour, the distance from the frame to the surface of the water should again be measured. The leakage should be calculated using volumes.

The leakage at the end of the test period should not exceed the maximum allowable calculated for the test section. In accordance with OPSS 410 allowable leakage is calculated as 0.075 L/mm diameter/100 metres of sewer pipe/hr (8.1 US gal/inch diameter/mile of sewer pipe/hr).

An allowance of 3.0 liters per hour per metre of head (0.24 US gal/hr/ft of head) above the invert for each manhole included in the test section should be made.

Manhole should be tested separately if the test section fails.

5.7.12 Design Calculations

A tabular form may be used for recording sewer design calculations including required capacity, sewer size, sewer slope, roughness coefficient used, pipe capacity provided, flow velocity when flowing full, depth of flow, and actual flow velocity at peak flow if depth of flow is less than 0.3 of the pipe diameter. Typical sanitary and storm sewers design sheets are shown in Figures 5-1 and 5-2.

5.7.12.1 Air Test

The designer may consider the use of an air test where water is not readily available or the differential head in the test section is greater than 8 m (26 ft) or freezing temperatures exist. The air test should, as a minimum, conform to the test procedure described in OPSS 410.

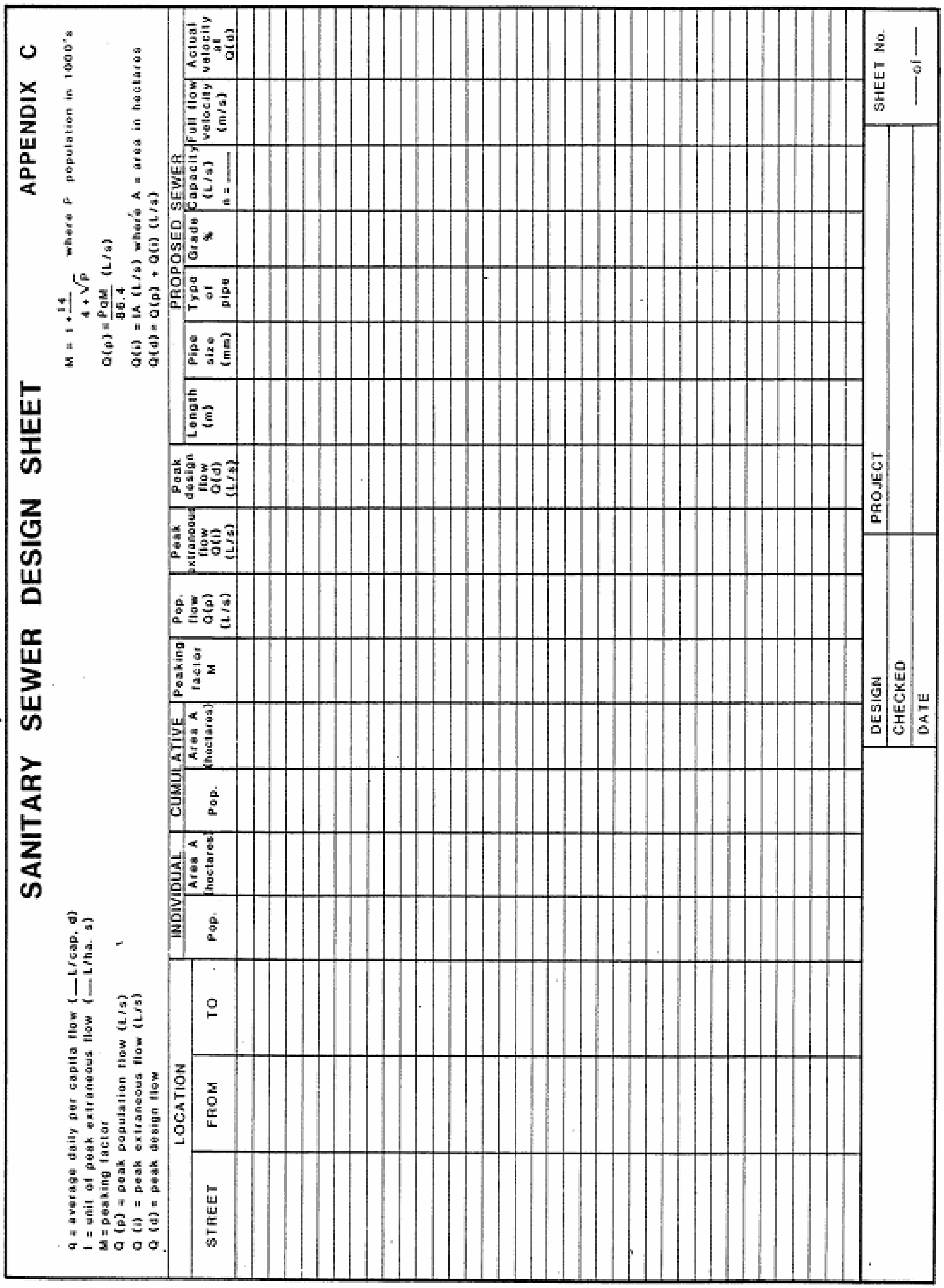
5.7.13 Bypasses and Overflows

Bypasses are considered to be flows that are diverted from the system, but not discharged to the environment. In the case of sewers this could include diversion to holding tanks or other sewers. Overflows are sewage flows that are diverted from the sewer and discharged directly to the environment. For additional details see *Section 8.5.6 - Bypasses and Overflows*.

5.7.14 Foundation Drainage

The connection of foundation drains to a sanitary sewer system is strongly discouraged by the ministry because of the serious negative hydraulic impacts that such connections can have on the sewer system and the potential hydraulic overloading of the sewage treatment plant (*Section 5.4.7 - Foundation Drainage*).

Figure 5-1 - Typical Sanitary Sewer Design Sheet



SANITARY SEWER DESIGN SHEET

APPENDIX C

q = average daily per capita flow (__L/cap. d)
I = unit of peak extraneous flow (__L/ha. s)
M = peaking factor
Q (p) = peak population flow (L/s)
Q (i) = peak extraneous flow (L/s)
Q (d) = peak design flow

$M = 1 + \frac{14}{4 + \sqrt{P}}$ where P population in 1000's

$Q(p) = \frac{PqM}{86.4}$ (L/s)

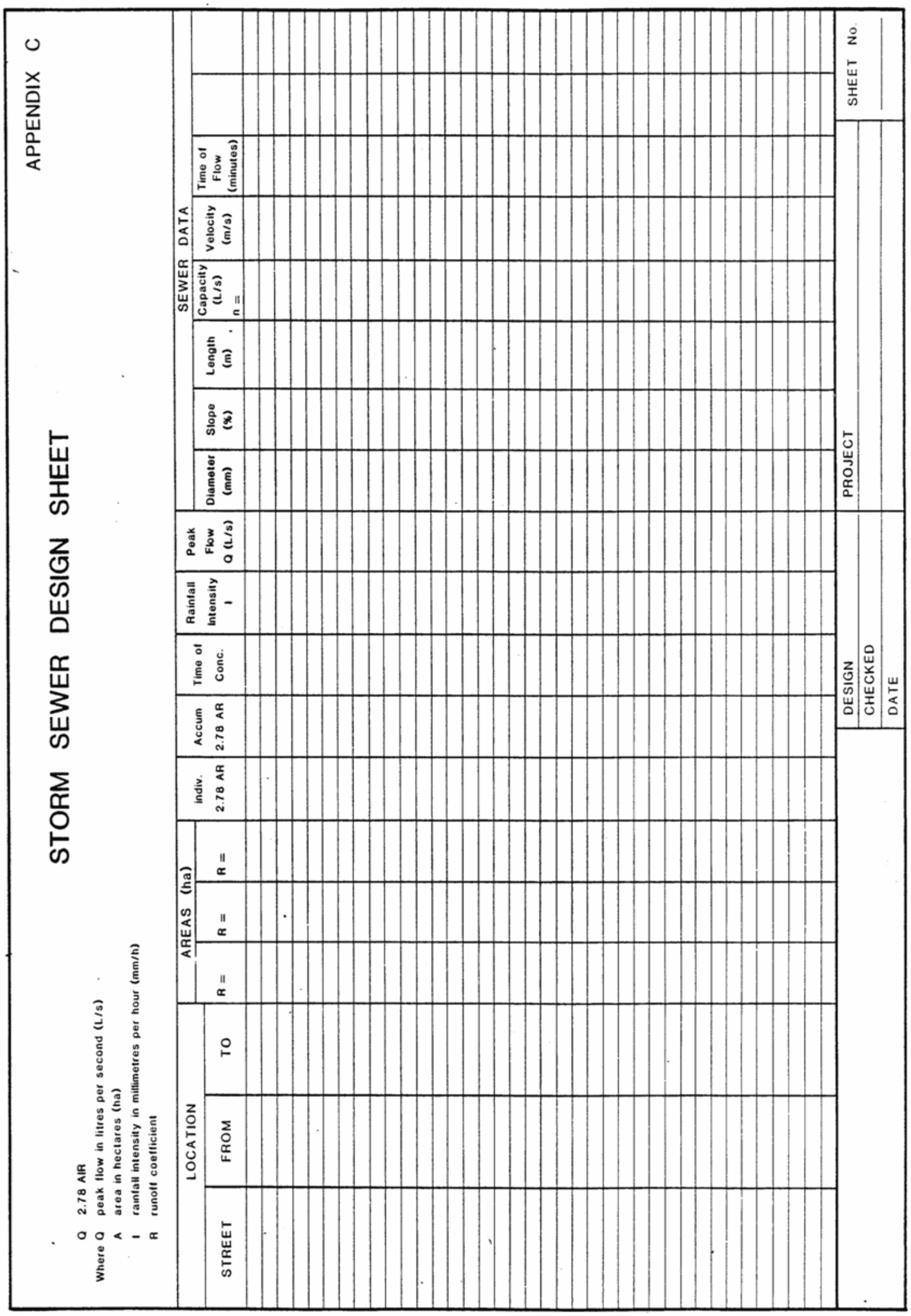
Q(i) = IA (L/s) where A = area in hectares

Q(d) = Q(p) + Q(i) (L/s)

LOCATION			INDIVIDUAL		CUMULATIVE		Peaking factor M	Pop. flow Q(p) (L/s)	Peak extraneous flow Q(i) (L/s)	Peak design flow Q(d) (L/s)	PROPOSED SEWER						
STREET	FROM	TO	Pop.	Area A (hectares)	Pop.	Area A (hectares)					Length (m)	Pipe size (mm)	Type of pipe	Grade %	Capacity (L/s) n = ___	Full flow velocity (m/s)	Actual velocity at Q(d)

DESIGN	PROJECT	SHEET No.
CHECKED		
DATE		___ of ___

Figure 5-2 - Typical Storm Sewer Design Sheet



APPENDIX C

STORM SEWER DESIGN SHEET

Q 2.78 AIR
Where Q peak flow in litres per second (L/s)
A area in hectares (ha)
I rainfall intensity in millimetres per hour (mm/h)
R runoff coefficient

LOCATION			AREAS (ha)								SEWER DATA							
STREET	FROM	TO	R =	R =	R =	indiv. 2.78 AR	Accum 2.78 AR	Time of Conc.	Rainfall Intensity I	Peak Flow Q (L/s)	Diameter (mm)	Slope (%)	Length (m)	Capacity (L/s) n =	Velocity (m/s)	Time of Flow (minutes)		

DESIGN
CHECKED
DATE
PROJECT
SHEET No.

5.8 ALTERNATIVE INSTALLATION AND CONSTRUCTION TECHNOLOGIES

5.8.1 Horizontal Directional Drilling

Horizontal Directional Drilling (HDD) is a trenchless construction method that uses guided drilling for boring a tunnel with an arc profile. This technique is used for long-distance crossings such as under rivers, lagoons, landfills or highly urbanized areas. The process involves three main stages. The first stage is to drill a pilot tunnel. This small tunnel is then reamed in stages until it is large enough to facilitate the final piping. The pipe is then installed by being pulled back through the prepared tunnel.

The HDD technique can be used in various ground conditions including hard rock, sand, silt and clay formations. The bore hole is supported by drilling fluid which mainly consists of bentonite, which has several functions including transport of cuttings, cooling off the drill bit, sealing and supporting the drilled hole and lubrication to reduce friction during pullback.

5.8.2 Micro Tunnelling

Micro tunnelling is typically used for pipes ranging from 450 to 1400 mm (NPS-18 to NPS-56) in diameter and 12 to 15 m (40 to 50 ft) in depth. Micro tunneling is accomplished using a remote-controlled boring machine to carve out a tunnel and install a new pipe in a nearly simultaneous process.

Micro tunneling causes minimal disruption to surface activities. It is particularly effective where piping is required in below-groundwater conditions, in unconsolidated soils and where above- and below-ground obstructions exist.

5.8.3 Pipe Bursting

Pipe bursting is a semi-trenchless method of installing factory-manufactured pipes and service connections in place of severely damaged water and sewer pipes. It is an ideal solution for upsizing capacity of existing sewer lines ranging from 150 to 900 mm (NPS-6 to NPS-36) in diameter.

Pipe bursting is a good solution for installing new pipes in high-profile areas where disruption to surroundings is an important consideration, because it requires minimal excavation.

In the pipe bursting process, a new polyethylene pipe is pulled through an old pipeline of equal or smaller size. The old pipeline is shattered as the new pipe is pulled through, with the pieces displaced into the surrounding soil. The pipe bursting process is unique in allowing enlargement of existing sewers with minimal excavation.

5.9 MANHOLES

5.9.1 Location and Spacing

Manholes should be installed:

- At the end of each line;
- At all changes in grade, size, or alignment;
- At all intersections (except curvilinear sewers); and
- At distances not greater than 120 m (400 ft) for sewers of 375 mm diameter (NPS-15) or less and 150 m (500 ft) for sewers of 450 mm diameter (NPS-18) to 750 mm diameter (NPS-30), except that distances up to 185 m (600 ft) may be considered in cases where adequate modern cleaning equipment for such spacing is provided.

Greater spacing may be permitted for larger sewers. Cleanouts may be used only for special conditions and should not be substituted for manholes nor installed at the end of laterals greater than 45 m (150 ft) in length.

5.9.2 Drop Type

A drop pipe should be provided for a sewer entering a manhole at an elevation of 610 mm (24 in) or more above the manhole invert. Where the difference in elevation between the incoming sewer and the manhole invert is less than 610 mm (24 in), the invert should be filleted or benched to prevent solids deposition.

Drop manholes should be constructed with an outside drop connection. Inside drop connections (when necessary) should be secured to the interior wall of the manhole and provide access for cleaning.

The entire outside drop connection should be encased in concrete due to the unequal earth pressures that would result from the backfilling operation in the vicinity of the manhole.

5.9.3 Diameter

The minimum diameter of manholes should be 1200 mm (48 in); larger diameters are preferable for large diameter sewers. A minimum access diameter of 610 mm (24 in) should be provided.

5.9.4 Flow Channel

The flow channel straight through a manhole should be made to conform as closely as possible in shape and slope to those of the connecting sewers. The channel walls should be formed or shaped to the full height of the crown of the outlet sewer in such a manner to not obstruct maintenance, inspection or flow in the sewers.

When curved flow channels are specified in manholes, including branch inlets, minimum slopes indicated in Table 5.4 should be increased to maintain acceptable velocities.

5.9.5 Bench

A bench should be provided on each side of any manhole channel when the pipe diameter is less than the manhole diameter. The bench should be sloped no less than 40 mm/m (½ inch per foot or 4 percent). No lateral sewer, service connection, or drop manhole pipe should discharge onto the surface of the bench.

5.9.6 Water Tightness

Manholes should be of the pre-cast concrete or cast-in-place concrete type. Manhole lift holes and grade adjustment rings should be sealed with non-shrinking mortar.

Inlet and outlet pipes should be joined to the manhole with a gasketed flexible watertight connection that allows differential settlement of the pipe and manhole wall to take place.

Watertight manhole covers are to be used wherever the manhole tops may be flooded by street runoff or high water. Locked manhole covers may be desirable in isolated easement locations or where vandalism may be a problem.

5.9.7 Inspection and Testing

The specifications should include a requirement for inspection and testing for watertightness or damage prior to placing into service. Air testing, if specified for concrete sewer manholes, should conform to the test procedures described in ASTM C 1244.

5.9.8 Access

Manhole steps should be 400 mm (16 in) aluminum or galvanized rungs and should be provided at a spacing of 300 to 400 mm (12 to 16 in).

Safety chains should be provided on the downstream side of manholes for sewers larger than 1200 mm in diameter (NPS-48).

Safety landings should be provided in accordance with Ontario Regulation 632/05 *Confined Spaces*, made under the *Occupational Health and Safety Act* (OHSA).

5.9.9 Corrosion Protection for Manholes

Where corrosive conditions due to septicity or other causes are anticipated, corrosion protection on the interior of the manholes should be provided.

5.9.10 Frost Straps

Frost straps should be provided to hold pre-cast manhole sections together. In areas where the freezing index is greater than 500 freezing degree days Celsius, pre-cast manholes/chambers should have three steel straps extending vertically from top to bottom and held by bolts in the top and bottom sections.

When the design freezing index equals or exceeds 1800 freezing degree days Celsius, an additional granular water draining layer at least 0.3 m (12 in) thick should surround the manhole.

The designer should refer to *Chapter 6 - Challenging Conditions Affecting Servicing* for further details.

5.10 PIPE DESIGN

5.10.1 Pipe Strength Requirements

Sewer pipes selected for any particular application should be able to withstand, with an adequate margin of safety, all the combinations of loading conditions to which it is likely to be exposed.

Pipes used in gravity flow sewers are usually not subjected to internal pressure, except to a small degree under conditions of surcharge. Therefore, in the design of sewer pipes, internal pressure is usually not a significant factor. In special cases involving excessive surcharge, such as in inverted siphons, pressure pipe may be required.

Sewer pipe installed in a backfilled trench carries the external static, live and hydraulic loads placed on it. The factor of external load is very important in the design of sewer pipes, regardless of the material used.

The design procedures to be used to calculate earth loading, superimposed loads, and the supporting strength of sewer pipe under various types of installations and bedding conditions are covered in pipe supplier's catalogues or design manuals such as Water Pollution Control Federation (now Water Environment Federation), *Design and Construction of Sanitary and Storm Sewers*, Manual of Practice MOP-9.

5.10.2 Inverted Siphons

Inverted siphons should have at least two barrels, with a minimum pipe size of 150 mm (NPS-6). They should be provided with necessary appurtenances for maintenance, convenient flushing and cleaning equipment. The inlet and discharge structures should have adequate clearances for cleaning equipment, inspection and flushing. Design should provide sufficient head and appropriate pipe sizes to secure velocities of at least 0.9 m/s (3.0 ft/s) for design average daily flows. The inlet and outlet details should be so arranged that the design average daily flow may be diverted to one barrel and so that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance. Siphon pipes and chambers, when subject to hydrostatic uplift forces, should have sufficient weight or anchorage to prevent their flotation when empty.

5.11 SEWER SYSTEM REHABILITATION

5.11.1 General

The selection of a rehabilitation method should be based on a detailed analysis of existing sewer system conditions, including:

- Identification of the need for rehabilitation;
- Evaluation of the physical installation;
- Assessment of expected performance attributes and requirements of potential rehabilitation technologies; and
- Analysis of costs.

Use any one or a combination of several analysis techniques to determine the need for pipe systems.

5.11.2 Flow Analysis

The designer should perform flow analyses to determine whether the existing or rehabilitated conduit will have sufficient capacity to accommodate the required flows. Flow monitoring should provide quantitative data that establish existing average and peak flows as well as the existence and magnitude of infiltration or inflow. Existing flow analysis and future flow forecasts should also be performed to determine the required hydraulic capacity of the pipeline. If insufficient capacity exists, and regardless of the beneficial hydraulic impacts of potential rehabilitation methods, alternative improvement possibilities should be explored.

5.11.3 Sewer Pipe Analysis

The principal method used to evaluate the physical condition of an existing sewer pipe is closed-circuit television (CCTV) systems. Television inspection should be performed using high-resolution, colour, video equipment with accurate distance measurement. A record of the inspection should be kept on a videotape, with headings noting the location, date and firm performing the work. The distance meter should have an accuracy of 60 mm (2.4 in) over the length of the sewer.

The television camera should travel through the sewer line in either direction at a speed no greater than 10 m/min (30 ft/min), stopping as necessary to ensure proper documentation of the sewer's condition. Written logs should document the video, estimating rates of infiltration and describing the internal condition of the pipeline.

Flow should be minimized or suspended altogether to provide the most comprehensive observations possible. For short reaches in smaller pipelines, it may be possible to briefly stop flow using sewer plugs without causing pipeline surcharging. On larger-diameter pipelines, it may be possible to perform the internal inspection by using hand-held video recording equipment.

Prior to internal observation, it is necessary to clean the host pipeline so that a complete scope of existing conditions can be viewed and a suitable technique can be chosen for performing rehabilitation procedures.

Cleaning requirements and the level of effort may vary depending on pipe age, slope, service and physical integrity. During cleaning, liquids and solids removed from pipelines should be analyzed to identify any hazardous components. Disposal methods and locations of such hazardous components should also be determined.

5.11.4 Temporary Flow Management

Existing or potential flows should be managed during internal inspection, cleaning and rehabilitation activities. A flow management plan should be prepared to ensure that the work program is not adversely affected. Flow management requirements may vary greatly depending on the type of system being evaluated or rehabilitated, separate sanitary, storm sewer systems, combined sewer systems or process pipelines.

A number of different flow management methods may be used to satisfy work program needs. These methods include:

- Flow division through parallel or other available piping systems;
- Stoppage of flows for the duration of the procedure when sufficient storage exists in upstream facilities; and
- Bypassing flows around the work site.

In all instances, all flow sources should be considered as well as their magnitude, frequency and timing when selecting an appropriate flow management method. Contingency plans should be developed to identify backup procedures for coping effectively with extreme or adverse conditions.

5.11.5 Safety

Pipeline rehabilitation involves personnel entering into confined spaces, many of which have potentially hazardous atmospheres. Therefore, the contract document should require implementation of safety plans by all contractors and subcontractors involved. These safety plans should address procedures for confined space entry, including permitting and all other appropriate regulatory requirements established by the *Occupational Health and Safety Act* (OHSA) and other applicable federal, provincial and local regulations.

5.11.6 Service Connections

Prior to rehabilitation, both active and inactive service connections should be identified using dye testing and other complementary procedures. Active services should be restored using appropriate rehabilitation methods and inactive services may be sealed, if desired.

5.11.7 Sewers and Manholes Rehabilitation

Selection of a sewer rehabilitation technique depends upon several factors, including the condition of the host pipe, hydraulic capacity both before and after rehabilitation and number of service laterals. Basic trenchless sewer rehabilitation methods include slip lining, cured-in-place products, segmental pipe lining, spiral wound pipe lining, close-fit pipe lining, pipe bursting and pipe removal (micro tunneling) and lining. Where applicable, rehabilitation methods may require simple repairs and maintenance such as grouting and sealing, or point repairs.

Rehabilitation of sewer manholes may be required because of excessive leakage or structural concerns. Repairs may include using pressure pointing, chemical grouting to repair critical spots, renovating the whole structure by monolithic surfacing techniques or by installing a new manhole.

5.12 STREAM CROSSINGS

5.12.1 Location

5.12.1.1 Cover Depth

The top of all sewers entering or crossing streams should be at a sufficient depth below the natural bottom of the stream bed to protect the sewer. In general, the following cover requirements should be met:

- 0.3 m (1 ft) of cover, where the sewer is located in rock;
- 0.9 m (3 ft) of cover, where the sewer is not located in rock. In major streams, more than 0.9 m (3 ft) of cover may be required; and
- In paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.

Less cover may be justified only if the proposed sewer crossing will not interfere with future modifications to the stream channel.

5.12.1.2 Horizontal Location

Sewers along streams should be located at a sufficient distance away from the stream, providing for future possible stream widening and to prevent pollution by siltation during construction.

5.12.1.3 Structures

The sewer outfalls, headwalls, manholes, gate boxes, or other structures should be located so they do not interfere with the free discharge of flood flows of the stream.

5.12.1.4 Alignment

Sewers crossing streams should be designed to cross the stream as nearly perpendicular to the stream flow as possible and should not change grade.

Sewer systems should be designed to minimize the number of stream crossings.

5.12.2 Construction

5.12.2.1 Materials

Sewers entering or crossing streams should be constructed of ductile iron pipe with mechanical joints; otherwise, they should be constructed to remain watertight without changes in alignment or grade. Material used to backfill the trench should be stone, coarse aggregate, washed gravel, or other materials which will not readily erode, cause siltation, damage pipe during placement, or corrode the pipe.

5.12.2.2 Siltation and Erosion

Construction methods that minimize siltation and erosion should be employed. The design engineer should include in the project specifications the methods to be employed in the construction of sewers in or near streams. Such methods should provide adequate control of siltation and erosion by limiting unnecessary excavation, disturbing or uprooting trees and vegetation, dumping of soil or debris or pumping silt-laden water into the stream. Specifications should require that cleanup, grading, seeding and planting or restoration of all work areas should begin immediately. Exposed areas should not remain unprotected for more than seven days.

5.13 AERIAL CROSSINGS

Support should be provided for all joints in pipes utilized for aerial crossings. The supports should be designed to prevent frost heave, overturning and settlement.

Precautions against freezing, such as insulation and increased slope, should be provided. Expansion joints should be provided between above ground and below ground sewers. Where buried sewers change to aerial sewers, special construction techniques should be used to minimize frost heaving.

For aerial stream crossings, the impact of flood waters and debris should be considered. The bottom of the pipe should be placed no lower than the elevation of the 50-year flood. Ductile iron pipe with mechanical joints is recommended.

5.14 PROTECTION OF DRINKING WATER SYSTEMS

When sewers are proposed in the vicinity of any drinking water system facilities, requirements of the ministry Guideline F-6, *Sewer and Watermain Installation: Separation Distance Requirement*, should be used to confirm acceptable separation distances.

There should be no physical connections between a drinking water system and a sewer or appurtenance which would permit the passage of any sewage or polluted water into the drinking water supply. No watermain should pass through or come into contact with any part of a sewer manhole.

5.14.1 Separation from Drinking Water System Facilities

Sewers, drains and similar sources of contamination should be kept at least 15 m (50 ft) from drinking water reservoirs below normal ground surface and groundwater well sources. Mechanically-jointed water pipes, pressure tested in accordance with OPSS 701, *Construction Specifications for Watermain Installation in Open Cut*, at a pressure of 350 kPa (50 psi) without leakage may be used for gravity sewers at lesser separation.

All existing drinking water system facilities, such as tanks, wells, or other treatment units, within 60 m (200 ft) of the proposed sewer should be shown on the plans.

Soil conditions in the vicinity of the proposed sewer within 60 m (200 ft) of drinking water system facilities should be determined and shown on the plans.

5.14.2 Separation from Watermains

5.14.2.1 Vertical Separation

When it is not practical to maintain a separate trench and a minimum horizontal separation distance, the crown of the sewer should be at least 0.5 m (1.6 ft) below the invert of the watermain and separated by *in situ* material or compacted backfill. Joints should be offset as much as possible between sewers and watermains.

Where this vertical separation cannot be obtained, the sewers should be constructed of watermain quality pipe, pressure tested in place at a pressure of 350 kPa (50 psi) without leakage in accordance with the OPSS 701.

In rock trenches, drainage should be provided to minimize the effects of impounding of surface water and/or the leakage from sewers in the trench.

5.14.2.2 Sewer and Watermain Crossings

Watermains should cross above sewers wherever possible. Whether the watermain is above or below the sewer, a minimum vertical distance of 0.5 m (1.6 ft) between the outside of the watermain and the outside of the sewer should be provided to allow for proper bedding and structural support of the watermain and sewer pipes. Sufficient structural support for the sewer pipes should be provided to prevent excessive deflection of the joints and settling. The length of water pipe should be centered at the point of crossing so that joints in the watermain will be equidistant and as far as possible from the sewer. The crossing should be perpendicular if possible.

When it is impossible to obtain proper horizontal and vertical separation as stipulated above, one of the following methods should be specified:

- The sewer should be designed and constructed equal to the water pipe and should be pressure tested at 350 kPa (50 psi) to assure watertightness; and
- Either the watermain or the sewer line should be encased in a watertight carrier pipe which extends 3 m (10 ft) on both sides of the crossing, measured perpendicular to the watermain.

5.14.2.3 Service Connections

Wherever possible, the construction practices outlined in *Section 5.14.2.1 - Vertical Separation* should be applied to sewer and water service connections.

5.14.2.4 Tunnel Construction

If a tunnel is of sufficient size to permit a person to enter it, a sewer and watermain may be placed through the tunnel providing the watermain is hung above the sewer. If the tunnel is sized only for the pipes or is subject to flooding, the sewers should be constructed of watermain quality pipe, pressure tested in place according to the OPSS 701 at a pressure of 350 kPa (50 psi) without leakage.

5.15 ALTERNATIVE SANITARY SEWER SYSTEMS

5.15.1 Applications

Each application has its own set of site-specific characteristics which may make an alternative sanitary sewer type attractive. The following are general design considerations only. For more information, the designer should refer to: Water Pollution Control Federation (now Water Environment Federation) *Alternative Sewer Systems*, Manual of Practice FD-12, Facilities Development (1986) and Environmental Protection Agency, Manual: *Alternative Wastewater Collection Systems*, EPA-625/1-91/024, Office of Research and Development (1991).

5.15.1.1 Population Density

When housing is sparse, resulting in long reaches between services, the cost of providing conventional sewers is often prohibitive. Pressure sewers, small diameter gravity sewers and vacuum sewers are typically less costly on a linear metre (foot) basis, so often prove to be more cost-effective when serving sparse populations.

5.15.1.2 Ground Slopes

If intermittent rises in the grade occur, conventional sewers may become cost-prohibitively deep. The small diameter, variable grade, low pressure sewage collection system in conjunction with septic tank effluent pumping (STEP system) can be economically applied. Vacuum sewers may be particularly adaptable to this topographic condition, as long as head requirements are within the limits of available vacuum.

In flat terrain, conventional sewers become deep due to the continuous downward slope of the main, requiring frequent use of sewage pumping stations. Both the deep excavation and the pumping stations are expensive. Small diameter gravity sewers (SDGS) are buried less deep, which may allow reduced excavation costs or fewer pumping stations. Pressure sewers or vacuum sewers are often found to be practical in flat areas.

In areas where the treatment facility or interceptor sewers are higher than the system, pressure sewers and vacuum sewers are generally preferred, but should be evaluated against SDGS systems with pumping stations.

5.15.1.3 Subsurface Obstacles

Where rock excavation or high groundwater levels are encountered, the shallow burial depth of alternative sewers reduces the amount of rock to be excavated or the need for dewatering.

5.15.2 Pressure Sewer Systems

Pressure sewers are small diameter pipelines buried just below frost level. These pipes can follow the profile of the ground because gravity flow is not needed. Each home connected to the pipeline requires either a grinder pump (GP) or a septic tank effluent pump. Main diameters typically range from 50 to 150 mm (NPS-2 to NPS-6) with service lateral diameters of 25 to 38 mm (1 to 1.5 in). Polyvinyl Chloride (PVC) is the most common piping material for these systems.

5.15.2.1 System Layout

Pressure sewer systems should be laid out taking the following into consideration:

- Branched layout rather than gridded or looped;
- Maintenance of cleansing velocities especially when grinder pump type pressure sewers are used;
- Minimizing high head pumping and downhill flow conditions;
- Locating on lot facilities close to the home for ease of maintenance; and
- Providing each home its own tank and pump.

The designer should also refer to *Section 7.9 - Forcemains* for further details.

5.15.3 Vacuum Sewer Systems

Vacuum sewer systems include a vacuum station, collection piping, sewage holding tanks and valve pits. In these systems, sewage from an individual building flows by gravity to the location of the vacuum ejector valve. This valve seals the line leading to the main sewer in order to maintain required vacuum levels. When a given amount of sewage accumulates behind the valve, the valve opens and then closes allowing sewage to enter the main sewer and be taken away. Vacuum pumps in a central location maintain vacuum in the system.

5.15.3.1 House Services

Each house on the system should have its own holding tank and vacuum ejector valve. Holding tank volume is usually 115 L (30 US gal). The level in the holding tank is monitored by a sensor. When the tank is filled, the valve is signaled to open. The valve stays open for an adjustable period of time and then closes. During the open cycle, the holding tank contents are evacuated. The time is usually set to hold the valve open for a total time equal to twice the time required to admit the sewage. In this manner, air is allowed to enter the system behind the sewage. The time setting is dependent on the valve location since the vacuum available will vary throughout the system, thereby governing the rate of sewage flow.

The valve pit is typically located along a property line and may be combined with the holding tank. These pits are usually made of fiberglass, although modified concrete manhole sections have been used. An anti-flotation collar may be required.

5.15.3.2 Collection Piping

The vacuum collection piping usually consists of 100 mm and 150 mm diameter (NPS-4 and NPS-6) mains. Smaller mains are not recommended as the cost savings are considered to be insignificant.

Rubber gasketed PVC pipe which has been certified by the manufacturer as being suitable for vacuum service is recommended. Solvent welding should be avoided if possible. The mains are generally laid to the same slope as the ground with a minimum slope of 0.2 percent. For uphill transport, lifts are placed to minimize excavation depth. There are no manholes in the system; however, access can be gained at each valve pit or at the end of a line where an access pit may be installed. Installation of the pipe and fittings should follow water distribution system practices. Division valves are installed on branches and periodically on the mains to allow for isolation. Plug valves and resilient wedge gate valves have been used.

5.15.3.3 Vacuum Station

Equipment in the station includes a collection tank, vacuum reservoir tank, vacuum pumps, sewage pumps and pump controls. Emergency power is vital to these systems and should be provided for in the design.

The collection tank should be made of either steel or fiberglass. The vacuum reservoir tank should be connected directly to the collection tank to prevent droplet carryover and to reduce the frequency of vacuum pumps starts. Vacuum pumps can be either liquid ring or sliding vane type and should be sized for a 3 to 5 hr/d run-time. The sewage discharge pumps should be non-clog type with sufficient *net positive suction head* (NPSH) to overcome tank vacuum. Level control probes should be installed in the collection tank to regulate the sewage pumps. A fault monitoring system to alert the system operator if low vacuum or high sewage level conditions occur should be incorporated into the design.

5.15.4 Small Diameter Gravity Sewers

SDGS systems are most cost-effective when housing density is low, the terrain has undulations of low relief and the elevation of the system terminus is lower than nearby servicing areas. They can also be effective in very flat areas, rocky or unstable soil conditions and in areas where there is a high water level. SDGS do not have a large amount of excess capacity, that is typical of gravity sewers, and should be designed to allow an adequate amount of future growth.

SDGS require preliminary treatment through the use of interceptor or septic tanks upstream of each connection. The tanks remove solids and allow for the sewer piping to be designed differently than conventional sewers which also carry the solids. Collector mains are smaller in diameter and can be laid with variable or inflective gradients. Fewer manholes are needed and most can be replaced with cleanouts except at major junctions to limit infiltration/inflow and entry of grit. The required size and shape of the mains is dictated primarily by hydraulics rather than solids carrying capabilities.

5.15.4.1 House Services

House connections are made at the inlet to the interceptor tank through which all household sewage enters the system. Interceptor tanks are buried, watertight tanks with baffled inlets and outlets. They are designed to remove both floating and settleable solids from the sewage through quiescent settling over a period of 12-24 hours. Ample volume should be provided for storage of the solids which should be periodically removed through an access port. Typically, a single-chamber septic tank, vented through the house plumbing stack vent, is used as an interceptor tank. Service laterals connect the interceptor tank with the collector main. They are usually 75 to 100 mm (NPS-3 to NPS-4) in diameter, but should be no larger than the collector main to which they are connected. They may include a check valve or other backflow prevention device near the connection to the main.

5.15.4.2 Collector Mains

Collector mains are small diameter plastic pipes with typical minimum diameters of 75 to 100 mm (NPS-3 to NPS-4). The mains are trenched into the ground at a depth sufficient to collect the settled sewage from most connections by gravity. Unlike conventional gravity sewers, SDGS are not necessarily laid on a uniform gradient with straight alignments between cleanouts or manholes. In places, the mains may be depressed below the hydraulic grade line. Also, the alignment may be curvilinear between manholes and cleanouts to avoid obstacles in the path of sewers.

Cleanouts, manholes and vents provide access to the collector mains for inspection and maintenance. In most circumstances, cleanouts are preferable to manholes because they are less costly and can be more tightly sealed to eliminate most infiltration and grit which commonly enter through manholes. Vents are necessary to maintain free flowing conditions in the mains. Vents in household plumbing are sufficient except where depressed sewer sections exist. In such cases, air release valves or ventilated cleanouts may be necessary at the high points of the main.

Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEP units or mainline lift stations may be used.

CHAPTER 6

CHALLENGING CONDITIONS AFFECTING SERVICING

CHAPTER 6

CHALLENGING CONDITIONS AFFECTING SERVICING

This chapter presents design guidelines, suggestions and ideas which may assist in the application and design of alternate technologies for underground servicing of areas that are affected by challenging conditions. For more detailed information on this subject the designer should refer to the ASCE/CSCE (1996), *Cold Regions Utilities Monograph*, (*formerly Cold Climate Utilities Deliver Design Manual, Environment Canada*) available from the ASCE bookstore website (*http://www.asce.org*).

6.1 GENERAL

Challenging conditions may be a result of: climate, geology, hydrogeology, area location (remoteness), topography or any combination of these factors. Challenging conditions are often associated with northern communities, but can also occur in urban areas where above-ground piping is necessary (i.e., bridge crossings or over permafrost) and/or shallow buried obstructions such as culverts cause pipe to be placed in the frost zone. Generally biological treatment processes operate most effectively at sewage temperatures above 10°C (41°F) and critical design conditions need to be assessed at low operating temperatures to ensure year round effluent objectives can be met. This low temperature evaluation is particularly important for nitrifying plants that have low ammonia effluent objectives.

6.2 CLIMATIC FACTORS

The main climatic elements that can affect low ground temperatures are cold air temperatures and the amount of snow cover. With below freezing temperatures, the designer should determine whether the conditions are such that sewage in the proposed municipal sewer pipe would freeze or glaciate and the pipe affected by frost heave may hinder gravity flow by forming pockets. Historical information on Ontario climate is available from the Environment Canada website *(http://www.climate.weatheroffice.ec.gc.ca).* Design temperature data are also provided in the Supplementary Standard SB-1 of the *Building Code* (O. Reg. 350/06) made under the *Building Code Act*, 1992. Other references of note for additional information include:

- Smith, DW and Hrudey, SE (1981), *Design of Water and Wastewater Services for Cold Climate Communities*; and
- Air Force (1987), *Technical Manual - Arctic and Subarctic Construction Utilities*, AFR 88-19, Volume 5.

The main indicator utilized to determine the relative "air coldness" of an area is the "Freezing Index". The "Freezing Index" is defined as the number of degree days [above and below 0ºC (32°F)] between the highest point in the autumn and the lowest point the next spring on the cumulative degree-day time curve for one freezing season. It is recommended that the designer consider the coldest month.

The climatic factor most seriously impacting the design, cost and operation of sewer pipes is frost. The depth to which it penetrates depends upon the Freezing Index as well as the frost susceptibility and the thermal conductivity of the soil. The designer should refer to the Environment Canada, *Cold Climate Utilities Delivery Design Manual* (Environmental Protection Service Report No. EPS 3-WP-79-2).

Another factor to be considered is frost heave. As the water in the pores of the soil freezes, there is an associated increase in the volume of the soil of up to 5%. If ice lenses form in the soil, much greater volume increases may occur. Any *sewage works* that is to be constructed within the frost zone should be designed with consideration given to the rise of the ground surface due to frost action.

For more information regarding frost and freezing, the designer should refer to the National Research Council of Canada, Institute for Research in Construction (*http://www.irc.nrc-cnrc.gc.ca*) and the Ontario Ministry of Transportation.

6.3 GEOLOGICAL FACTORS

The predominant geological factor which can have an effect on the design of sewage works is the presence of rock and its proximity to the surface. This phenomenon is common in many areas of the Province and predominant in Northern Ontario where the main geological feature is the Precambrian formation of the Canadian Shield.

Other factors concerning the geology of the northern parts of the Province that affect the design of sewer systems are:

- The presence of muskeg which can be found in depths varying from less than 0.3 m (1 ft) to in excess of 3.0 m (10 ft);
- Soil classification and frost susceptibility;
- Soil thermal conductivity;
- Soil chemistry (i.e., acidic and alkali soils); and
- The presence of a high water table.

6.4 REMOTE LOCATION

In certain regions of the Province, the mere location of the community to be serviced may be a factor in the design. Access to the site may be difficult, limited and/or expensive due to the lack of adequate road or rail transportation.

Access problems can affect for example the supply of materials, construction equipment, replacement parts and servicing. The designer should ensure that the servicing methods are adapted to suit local conditions. If special fittings and accessories are required that may be difficult to obtain, replace and service, this should be considered at the design stage and spares purchased during construction.

6.5 PERMAFROST

Permafrost is defined as soil, bedrock or other material that has remained below 0ºC (32°F) for two or more years. Continuous permafrost occurs in areas that are underlaid by permafrost with no thawed areas. Discontinuous permafrost occurs in an area underlain mostly by permafrost but containing small areas of unfrozen ground.

In Ontario, a state of discontinuous permafrost exists north of the line drawn from the southern tip of Hudson Bay, westerly to the point where the 53ºN parallel intercepts Ontario's western boundary, to the 55ºN parallel. More information on the distribution of permafrost in Ontario is available from Natural Resources Canada (*http://www.atlas.nrcan.gc.ca*).

Passive construction is usually used in permafrost conditions. This maintains the state of frozen permafrost by constructing insulated municipal sewer pipes. Permafrost conditions will not likely be met in any but the most remote northern areas of the Province.

6.6 DIFFICULTIES ASSOCIATED WITH CONVENTIONAL PRACTICES

Conventional design practice of installing sanitary sewers is to provide gravity flow from the basement of the building being serviced to a gravity collector system. At the high point in the system the collector sewer is generally in the order of 2.5 metres (8.2 ft) below grade. This depth is, in most instances, adequate to prevent freezing/glaciating of the sewage and to prevent any pockets from forming in the gravity sewers due to frost heave.

In areas where little or no overburden exists (i.e., rock), it is the practice to blast the required trench and remove the rock. The fragmented rock is often returned to the trench after a "protective" layer of cover material is placed over the pipe. The large voids present in rock backfill are conducive to greater frost penetration. Replacement of the rock backfill with granular backfill is recommended. In either case, the high thermal conductivity of the surrounding rock may be the governing design factor and should not be forgotten. This situation often necessitates installation of the sewers below the depth normally required for gravity flow from the basements of the buildings being serviced.

In areas with a high water table, infiltration into the system through pipe and manhole joints and private services can be a problem in many installations.

In some areas, the development density of the lots may be so low as to make sanitary sewer servicing infeasible due to the high installation costs per lot. In such areas, the use of septic tanks has been practiced. However, in an area where the presence of rock associated with very little overburden (e.g. less than 600 mm (24 in)) is the predominant feature, the use of septic tanks is not practical.

6.7 RETROFITTING OF EXISTING SEWERS

Frost can exert considerable load on a buried conduit and frost action can separate the sections of a precast manhole if the sections are not strapped.

There would appear to be little that can be done to an existing sewer system to protect it from the increased loads due to frost, although some studies suggest that slab insulation above the pipe will distribute the load. This may be considered in situations where the cover over an existing sewer is reduced due to the reconstruction of the road or re-grading.

Where existing manholes have separated and are permitting extraneous flows, the manholes should be grouted and/or reset and straps installed on the inside of the manhole to prevent any further movement.

In areas where the freezing index is greater than 500 freezing degree-days Celsius (932 degree days Fahrenheit), precast manholes/chambers should have three (3) steel straps extending vertically from top to bottom and held by bolts in the top and bottom sections.

The three steel straps should be located at points equidistant around the circumference of the manhole/ chamber.

When the design freezing index equals or exceeds 1,800 freezing degree-days Celsius (3272 degree-days Fahrenheit), an additional granular water draining layer at least 0.3 m (1 ft) thick should surround the manhole.

There are less significant problems with freezing of gravity sewers, service connections or forcemains in comparison to watermains and water services.

6.8 ALTERNATIVE DESIGN PRACTICES

In general, the costs of installing sanitary sewer services increases as the depth of burial increases. In areas that are subject to the effects of adverse conditions (such as the presence of rock, extreme frost or a high water table) the costs would be much greater as the depth to which these services need to be installed increases. The designer is referred to Chapter 12 in the *ministry* Design Guidelines for Drinking-Water Systems for information on water services in challenging conditions.

6.8.1 Thermal Considerations

When dealing with services and/or forcemains that are located in the active frost zone, it is possible to reduce heat loss and increase time before freezing by using pre-insulated piping with or without electric heat tracing.

6.8.2 Shallow Buried Pre-Insulated Servicing Systems

"Shallow buried" means a system that is partially or totally within the frost zone (i.e., cover only for physical protection) and "insulated" means reducing the heat loss from the pipe by applying various amounts of insulation to the buried pipe with or without heat tracing.

The specification for the work should indicate that "factory fabricated, pre-insulated, flexible piping system" is required.

6.9 SANITARY SEWER SYSTEMS

The fundamental concepts of sanitary sewer system design, including such items as layout and appurtenances, should follow the recommendations contained in *Chapter 5 - Design of Sewers*.

In the laying out of gravity sanitary sewers in areas where frost depth penetrations are great and/or other adverse conditions exist, the designer should consider alternate routing of the sewers (i.e., off the traveled/plowed portion of the road).

It is recommended that all gravity sewer system designs include the provision of frost straps on all manholes and that all pipe designs reflect the increased loading that will be experienced through either improved bedding or increased pipe strength, due to the increase in weight on the pipe.

Other manhole design considerations include:

- Use of a plastic film around the outside of the manhole to prevent bonding of the soil to the structure and damage from frost heaving;
- Use of manhole insulation, generally 75 mm (3 in) of polystyrene or urethane; and
- An insulated manhole cover to further reduce heat loss.

Attention to service connections should include consideration for insulation, heat tracing and flexible connections where damage due to freezing or frost heave is a concern.

Forcemains need to maintain an adequate scouring velocity and be able to drain between pumping cycles in cold climates. This can be accomplished by an electrically operated ball valve in the line to allow drainage back into the wet well between pump cycles.

The soil conditions such as rock or high *groundwater* can significantly affect the cost of the sewers and, in some instances, contribute to extraneous flow problems. It may be advisable in such circumstances to design and construct the system such that gravity drainage is only provided for the first floor and up. In assessing such an alternative, it is essential that the following factors be considered:

- The presence or absence of basements in the existing dwellings;

- The extent of "finishing " in an existing basement;
- The presence or absence of fixtures in the basement; and
- The need for a solids handling sewage pump in the basement, should the basement contain fixtures.

6.9.1 Alternate Sewage Collection Systems

In the recent past, several alternate methods of communal servicing have been introduced in Ontario and other jurisdictions with success. The designer should refer to *Section 5.15 - Alternative Sanitary Sewer Systems* for more details. If a previously untried sewage collection system is being proposed, the designer should refer to *Section 3.9 Technology Development* for guidance.

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CHAPTER 7

PUMPING STATIONS

CHAPTER 7

PUMPING STATIONS

This chapter covers the design of sewage pumping stations and forcemains. General requirements such as location, types, flows, reliability, emergency operations, controls and alarms and other special design details for pumping stations are presented. Design flow criteria are provided for pumping stations serving sanitary sewer systems. The design flow criteria for pumping stations serving combined sewer systems are identified where applicable. Note that the design flows are defined in *Section 8.3 - Definition of Terms.*

7.1 GENERAL

7.1.1 Station Capacity

Sewage pumping stations serving sanitary sewer systems should be able to pump the design peak instantaneous sewage flow. Sewage pumping stations should be designed so that they can be upgraded to handle future peak flows from the ultimate tributary area with minor modifications (e.g. pumps, motors or impeller changes). An economic evaluation may show that there are no savings by initially providing the design peak instantaneous *capacity*, then increasing the capacity at a later date. It is preferred that the ultimate anticipated peak flows from the tributary area could be handled with the addition of another pump and/or forcemain and other modifications. Oversizing the pumping station may impact station operations and should be evaluated during the design.

The capacity of a sewage pumping station serving a combined sewer system should be designed to pump all of the dry weather flow (DWF) plus 90% of the volume resulting from the design peak wet weather flow (WWF) that is above the design DWF (for an average year flow) to satisfy the requirements of *ministry* Procedure F-5-5, *Determination of Treatment for Municipal and Private Combined and Partially Separated Sewer Systems*. Higher pumping capacity or other control measures may be needed for swimming and bathing beaches affected by the overflow associated with the pumping station serving combined sewers in accordance with Procedure F-5-5.

7.1.2 Flooding

Sewage pumping station structures and electrical and mechanical equipment should be protected from physical damage by the 100-year design flood event. Sewage pumping stations should remain fully operational and accessible during the 25-year flood event. Regulations/requirements of municipalities, provincial and federal agencies regarding flood plain obstructions should be considered.

7.1.3 Accessibility and Security

The pumping station should be readily accessible by maintenance vehicles during all weather conditions. The facility should be located off the traffic way of streets and alleys. It is recommended that security fencing and access hatches with locks be provided.

7.1.4 Grit

Where it is necessary to pump sewage prior to grit removal, the designer should give special consideration to the wet well and pump station piping design to avoid operational problems from the accumulation of grit. This can include divided wells (i.e., for isolation and cleaning), aerated wells (i.e., to keep grit in suspension), steep wet well sides (i.e., to reduce area for accumulation) and desludging valves on pumps (i.e., to re-suspend and pump grit).

At some pump stations it may be beneficial to use bar screens, grinders, or comminutor devices. Design of bar screen facilities should include odour control and a method for handling the screenings.

7.1.5 Safety

Sewage pumping stations should be designed in such a manner as to ensure the safety of the operators and maintenance staff in accordance with the *Confined Spaces Regulation* (Ontario Regulation, or O.Reg. 632/05) made under the *Occupational Health and Safety Act* (OSHA). Typically, the following points should be considered:

- Any moving equipment should be covered with suitable guards to prevent accidental contact;
- Equipment that starts automatically should be suitably signed to ensure that operators are aware of this situation;
- Local lockouts on all equipment should be supplied so that maintenance personnel can ensure that they are completely out of service;
- Provision of fire/smoke detectors, fire extinguishers and sprinkler systems (where appropriate);
- All stairways and walkways should be properly designed with guardrails; and
- Confined spaces should be minimized, where applicable.

It is prudent to discuss risk management issues with the utility insurer to design safety and equipment components with their input and thus reduce long term risk. Also see *Section 8.9 - Safety*.

7.2 DESIGN

7.2.1 Types of Pumping Stations

There are four major types of sewage pumping stations that the designer may consider for site specific conditions: wet well/dry well, submersible, suction lift and screw pump. In a wet well/dry well pumping station, the pumps are located below grade in a dry well immediately adjacent to the wet well. In submersible pumping stations, submersible pumps are used in order to locate the pumps in the wet well, with the motor control centre mounted above-grade. Some submersible pumping stations come as a factory assembled unit with two pumps and motors pre-configured in a single well. Suction-lift pumping stations incorporate self-priming pumps in order to locate the pumps above the water level and either eliminate or decrease the depth of the dry well. Screw lift pumping stations use an Archimedean screw with the motor mounted above grade.

7.2.2 Structures

Dry wells, including their superstructure, should be completely separated from the wet well and common walls need to be gas tight.

Provision should be made to facilitate the removal of pumps, motors and other mechanical and electrical equipment. Individual pump and motor removal should not interfere with the continued operation of remaining pumps.

Suitable and safe means of access for persons wearing self-contained breathing apparatus should be provided to both dry wells and wet wells. Access to wet wells containing either bar screens or mechanical equipment requiring inspection or maintenance should conform with all safety requirements. Screens and other equipment located in pits more than 1.2 m (4 ft) deep need to be provided with stairway access.

Fresh air needs to be forced into the enclosed area (i.e., wet well). Where continuous ventilation is required at least 12 complete air changes per hour needs to be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour needs to be provided when personnel enter the area. Switches for operation of ventilation equipment should be marked and located conveniently. Explosion proof gas detectors need to be provided. Also refer to *Section 7.2.10 (Safety Ventilation)*.

For built-in-place pumping stations, a stairway to the dry well should be provided with rest landings at vertical intervals not to exceed 3.7 m (12 ft). For factory-built pump stations over 4.6 m (15 ft) deep, a rigidly fixed landing should be provided at vertical intervals not to exceed 3 m (10 ft). Where a landing is used, a suitable and rigidly fixed barrier should be provided to prevent an individual from falling past the intermediate landing to a lower level. A manlift or elevator may be used in lieu of landings in a factory-built station, provided emergency access is included in the design.

Where high *groundwater* conditions are anticipated, buoyancy of the sewage pumping station structures should be considered and, if necessary, adequate provisions should be made for protection.

Materials should be selected that are appropriate under conditions of exposure to hydrogen sulfide and other corrosive gases, greases, oils and other constituents frequently present in sewage. This is particularly important in the selection of metals and paints. Contact between dissimilar metals should be avoided or other provisions made to minimize galvanic action.

If more than one sewer enters a site, a junction manhole is preferred so that only one inlet to the wet well is required.

7.2.3 Pumps

Multiple pumps should be provided. Where only two units are provided, they should be of the same size, to provide a firm capacity with one unit out-of-service and at least capable of handling the 10-year design peak hourly flow. The designer should ensure that all pumps will be subjected to hydrostatic and operating tests performed by the manufacturer.

Pumps handling flow from combined sewers should be preceded by readily accessible bar racks to protect the pumps from clogging or damage. Where a bar rack is provided, a mechanical hoist is needed. Where the size of the installation warrants, mechanically cleaned and/or duplicate bar racks should be provided. The designer is referred to *Section 10.1 - Screening Devices* for more information.

Pumps handling sanitary sewage from 750 mm (30 in) or larger diameter sewers should also be protected by bar racks. Appropriate protection from clogging should also be considered for small pumping stations served by smaller sanitary sewers.

Except where grinder pumps are used, pumps handling raw sewage should be capable of passing spheres of at least 80 mm (3 in) in diameter. Pump suction and discharge openings should be at least 100 mm (4 in) in diameter.

The pump should be so placed that under normal operating conditions it will operate under a positive suction head, except where suction-lift pumps are used.

Each pump should be equipped with a time totalizer and provision for automatic or manual alteration of the lead pump.

Each pump should have an individual intake. Wet well and intake design should be such as to avoid turbulence near the intake and to prevent vortex formation.

A sump pump equipped with dual check valves should be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal leakage should be piped or channeled directly to the sump. The sump pump should be sized to remove the maximum pump seal water discharge that would occur in the event of a pump seal failure. See also *Section 7.6 - Alarm Systems.*

Pumping station designs should be based on system-head calculations and curves for three conditions using appropriate Hazen-Williams factor “C“ as follows:

(a) Low sewage level in the wet well, C = 120;

(b) Median sewage level over the normal operating range in the wet well, C = 130; and

(c) Overflow sewage level in the wet well, C = 140.

System-head curve (b) should be used to select the pump and motor since this will reflect the normal operating condition. The extreme operating ranges will be given by the intersections of curves (a) and (c) with the selected pump curve. The pump motor should be able to operate satisfactorily over this full range (i.e., between conditions (a) and (c)).

Although it is normal to size pumps and motors for design peak instantaneous flows, consideration should be given to how the future and ultimate sewage flow requirements can be handled. Ultimate sewage flows would account for the build-out of the catchments area. These operating points should also be shown on the system-head curves.

Where pumping stations are discharging directly to a sewage treatment plant or into a pumping station (i.e., forcemain directly into wet well of a downstream pumping station), some means of flow pacing is needed. This is provided most commonly by variable speed drives, depending upon the degree of flow pacing necessary. If even minor pump surges will have serious effects, variable speed pumps should be used. If small surges can be tolerated, two-speed or multiple speed pumps can be used.

The pumps and controls of main pumping stations and especially pumping stations discharging to or operated as part of a sewage treatment plant, should be selected to operate at varying delivery rates. In addition, where practical, such stations should be designed to deliver as uniform a flow as feasible in order to minimize hydraulic surges. The firm design capacity (with the largest unit out of service) of the pumping station serving sanitary sewers should be based on design peak instantaneous flow and should be adequate to maintain a minimum velocity of 0.6 m/s (2 ft/s) in the forcemain.

7.2.4 Electrical Equipment

Electrical systems and components (e.g. motors, lights, cables, conduits, switch boxes, control circuits) in raw sewage wet wells, or in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapours may be present, should comply with the *Electrical Safety Code* (O. Reg. 164/99) under the *Electricity Act, 1998* for Class I, Zone 1(old Division 1), Group D locations. In addition, equipment located in the wet well should be suitable for use under corrosive conditions. Each flexible cable should be provided with a watertight seal and separate strain relief. A fused disconnect switch located above ground should be provided for the main power feed for all pumping stations. When such equipment is exposed to weather, it should meet the requirements of the *National Electrical Manufacturers' Association* (NEMA) for weatherproof equipment NEMA 3R or 4. Lightning and surge protection systems should be considered. A 110-volt power receptacle to facilitate maintenance should be provided inside the control panel for lift stations that have control panels located outdoors. Ground Fault Circuit Interruption (GFCI) protection should be provided for all outdoor outlets.

Consideration should be given to the efficiency of the pumps, motors and drives to reduce energy requirements and cost. It is recommended that evaluation of such energy efficient units include both capital and operation and maintenance costs or life-cycle costs to provide an accurate evaluation of the benefits for such equipment.

7.2.5 Controls

Sewage level control sensing devices should be so located as not to be affected by turbulent flows entering the well or by the turbulent suction of the pumps. Bubbler type level monitoring systems should include dual air compressors. Provision should be made to automatically alternate the pumps in use. Suction-lift pump stations should be designed to alternate pumps daily instead of each pumping cycle to extend the life of the priming equipment. Float controls should be at least 300 mm (12 in) vertically and 450 mm (18 in) horizontally apart and positioned against a wall away from turbulent areas.

To minimize pumping costs and wet well depth, normal high sewage level (lag pump start elevation) may be designed to be above the invert of the inlet sewer(s), provided basement flooding and/or solids deposition would not occur. Where these problems cannot be avoided, the high sewage level (lag pump start elevation) should be approximately 300 mm (12 in) below the invert of the inlet sewer.

Low sewage level (pump shut down) should be at least 300 mm (12 in) or twice the pump suction diameter (D) above the centre line of the pump volute. The bottom of the wet well should be no more than D/2, nor less than D/3 below the mouth of the flared intake elbow.

7.2.6 Valves

Shutoff valves should be placed on the suction line of dry pit pumps.

Shutoff and check valves should be placed on the discharge line of each pump (except on screw pumps). The check valve should be located between the shutoff valve and the pump. Check valves should be suitable for the material being handled and should be placed on the horizontal portion of discharge piping except for ball checks, which may be placed in the vertical run. Valves should be capable of withstanding normal pressure and high-pressure transients.

All shutoff and check valves should be operable from the floor level and accessible for maintenance. Outside levers are recommended on swing check valves.

7.2.7 Wet Wells

Where continuity of pumping station operation is critical, consideration should be given to dividing the wet well into two sections, properly interconnected, to facilitate repairs and cleaning (including automatic cleaning devices). Divided wet wells should be considered for all pumping stations with firm capacities in excess of 100 L/s (1600 USgpm).

The design fill time and minimum pump-cycle time should be considered in sizing the wet well. The effective volume of the wet well should be based on design average daily flow and a filling time not to exceed 30 minutes unless the facility is designed to provide flow equalization. Other factors that should be considered include volumes required for pump-cycling, dimensional requirements to avoid turbulence problems, vertical separation between pump and control points, sewer inlet elevation(s), capacity required between alarm levels and basement flooding and/or overflow elevations, number of and horizontal spacing between pumps. The minimum surface plan area of a wet well should be 4.9 m^2 (53 ft^2) [i.e., 2.5 m (8.2 ft) diameter or 2.25 m (7 ft) square]. Wet wells should not provide excessive retention times, due to potential odour problems. For details of odour control the designer is referred to *Section 4.4 - Odour Control and Abatement Measures.*

The designer should ensure that easy and efficient removal of pumps, motors and other mechanical and electrical equipment is provided. A suitable and safe means of access for persons wearing self-contained breathing apparatus needs be provided to wet and dry wells and valve chambers. Equipment such as access hatches, ladders, service platforms, guards, grates and handrails, should be constructed of a suitable material when exposed to wet and/or corrosive conditions.

For pumping stations equipped with 50 kW (67 hp) or smaller pumps, the wet well should be of sufficient size to allow for a minimum cycle time of 10-minutes for each pump. To achieve this minimum detention time in a 2-pump station using constant speed pumps, the volume in cubic meters (m^3), between pump start and pump stop should be 0.15 times the pumping rate of one pump, expressed in L/s. For two-speed or variable speed pumps, pumps over 50 kW (67 hp), or for other numbers of pumps, the required volume depends on the operating mode of the pumping units. The pump manufacturer's duty cycle recommendations should be utilized in selecting the minimum cycle time. When the anticipated initial flow tributary to the pumping station is less than the design average daily flow, provisions should be made so that the fill time indicated is not exceeded for initial flows. When the wet well is designed for flow equalization, as part of a sewage treatment plant, provisions should be made to prevent septicity.

The wet well floor should have a minimum slope of 1 to 1 to the hopper bottom. The horizontal area of the hopper bottom should be no greater than necessary for proper installation and function of the inlet. The cross-sectional area of the wet well above the benching should be constant for the full depth of the wet well.

Access to the wet well should always be from the outside. An access ladder should be provided from the top of the slab to the service platform and a separated ladder from the platform to the bottom of the well.

The opening to the wet well should be no smaller than 750 by 900 mm (30 by 36 in), or 900 mm (36 in) in diameter. The cover should be equipped with a lock and pry lip and include a safety rail around the access. The opening edge should be flush with the vertical wall of the wet well. The opening to the wet well should be on the wall giving access to float controls, bubbler lines and similar equipment, without the necessity of entering the wet well.

The need for and type of screening facilities required for pumping stations varies with the characteristics of the sewage. For submersible pumping stations, screening may not be required, but for wet well/dry well stations, it is generally accepted practice to provide screening in the form of a basket screen or a removable bar screen. Although some basket screens may be cumbersome to remove and empty, their installation provides the advantage of not requiring entry of operating staff into the wet well for cleaning operations. With basket screens, guide rails should be tubular and similar to submersible pump rails. Manually cleaned bar screens should be sloped at 60º and have 38 mm (1.5 in) clear openings. The vertical sides should be solid. The minimum width should be 600 mm (24 in). A drain platform should be provided for screenings.

All wet wells need to be provided with ventilation. Natural ventilation will usually suffice for small pumping stations where access is limited. This can be achieved through two 100 mm (4 in) diameter vent pipes. Vents should be equipped with a gooseneck at the top, extending 900 mm (36 in) above the top of the slab of the wet well. The vents should be equipped with an insect screen. One vent pipe should extend within 0.3 m (1 ft) of the crown of the inlet sewer and the other should terminate on the underside of the roof slab. Natural ventilation can be supplemented with portable ventilation units. Adequate provisions for fresh air entry of all wet wells should be followed. In some cases mechanical ventilation may be preferred (see *Section 7.2.10 - Safety Ventilation*).

In wet well/ dry well installations, the air bubbler line (if used) and sump pump discharge should be raised above the overflow elevation and should cross between the wells below the frost line.

A service platform is normally required to allow for servicing of equipment and bar screen cleaning (if used).

7.2.8 Suction and Discharge Piping

Pump suction lines should be designed with the following features:

- Inlets consisting of 90° short radius down turned flared elbows;
- Suction velocities for 20-year or greater pumping requirements; preferably in low end of 0.8 to 2.0 m/s (2.6 to 6.6 ft/s) range;
- Flanged wall pipe with water stop collar;
- Gate valve (flanged);
- Flanged eccentric reducer; and
- Minimum pipe size of 100 mm (4 in).

Pump discharge piping should be designed with the following features:

- Velocities for the 20-year or greater sewage flow pumping needs, preferably in the low end of 0.8 to 4.0 m/s (2.6 to 13.1 ft/s) range;
- Flanged, concentric increaser;
- Spacer 150 to 300 mm (6 to 12 in) long with one flanged end and one grooved end for Victaulic coupling;
- Elbows (as necessary);
- Check valve (flanged), preferably horizontally placed;
- Gate valve (flanged);
- Flanged double branch elbow (for 2-pump station);
- Riser pipe; and
- Magnetic or other type of suitable flow meter and recorder (or pump timers for small, constant speed stations where accuracy of flow measurement is not critical - 3 timers minimum, one for each pump and one for pumps operating in parallel).

7.2.9 Dry Wells

The designer should also refer to information on dry wells in *Section 7.2.2 - Structures* and *Section 7.2.3 - Pumps*. Some additional design features are listed below:

- Ventilation, heating and dehumidification equipment should be provided to protect electrical control equipment from excess moisture;
- A lifting beam complete with permanently attached trolley or hook should be provided directly above the pump/motor assembly at a minimum height of 1.2 m (4 ft) above the motors to facilitate removal of the pump motors.

7.2.10 Safety Ventilation

Adequate ventilation should be provided for all pump stations. Where the dry well is below the ground surface, mechanical ventilation is needed. If screens or mechanical equipment requiring maintenance or inspection are located in the wet well, permanently installed ventilation is needed. There should be no interconnection between the wet well and dry well ventilation systems. Also, under no circumstances should wet well vents open into a building or connect with a building ventilation system.

In dry wells over 4.6 m (15 ft) deep, multiple inlets and outlets are desirable. Dampers should not be used on exhaust or fresh air ducts. Fine screens or other obstructions in air ducts should be avoided to prevent clogging.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment should be interconnected with the respective pit lighting system. Consideration should be given to automatic controls where intermittent operation is used. The manual lighting/ventilation switch should override the automatic controls. For a two-speed ventilation system with automatic switch over where gas detection equipment is installed, consideration should be given to increasing the ventilation rate automatically in response to the detection of hazardous concentrations of gases or vapours.

The fan wheel should be fabricated from non-sparking material. Automatic heating and dehumidification equipment should be provided in all dry wells. The electrical equipment and components should meet the criteria described in *Section 7.2.4 - Electrical Equipment.*

Wet well ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least 12 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Air should be forced into the wet well by mechanical means rather than solely exhausted from the wet well. The ventilating fan should be oriented to blow fresh air into the wet well at a point 900 mm (36 in) above the alarm level. The air change requirements should be based on 100 percent fresh air. Portable ventilation equipment should be provided for use at submersible pump stations and wet wells with no permanently installed ventilation equipment.

Dry well ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least 6 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. A system of two-speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic switch over to 6 changes per hour may be used to conserve heat. The air change requirements should be based on 100 percent fresh air.

Additional safety consideration can be given to installing cameras in dry well areas to allow for remote monitoring.

7.2.11 Flow Measurement

Suitable devices for measuring sewage flow should be provided at all pumping stations. Indicating, totalizing and recording flow measurement should be provided at pumping stations with a 75 L/s (1200 USgpm) or greater design peak hourly flow (DPHF). Elapsed time meters used in conjunction with annual pumping rate tests may be acceptable for pump stations with a DPHF of up to 75 L/s (1,200 USgpm) provided sufficient metering is configured to measure the duration of individual and simultaneous pump operation. Overflow volumes should be measured with instrumentation that can accurately monitor and continuously integrate the flow rate for an overflow event.

7.2.12 Water Supply

There should be no physical connection between any potable water supply and a sewage pumping station which under any condition might cause contamination of the potable water supply. The supply line should be equipped with a reduced pressure principle backflow preventer[1]. Where a potable water supply is to be used for human purposes (i.e., washrooms, sinks, showers, drinking fountains, eye wash stations and safety showers), a break tank, pressure pump and pressure tank needs to be provided. Water needs to be discharged to the tank through an air gap at least 150 mm (6 in) above the maximum flood line or the spill line of the tank, whichever is higher.

7.3 SUCTION-LIFT PUMPING STATIONS – SPECIAL CONSIDERATIONS

The designer should consider the applicable design recommendations provided in *Section 7.2 - Design* except as modified in this section.

7.3.1 Pump Priming and Lift Requirements

Suction-lift pumps should be of the self-priming or vacuum-priming type.

Self-priming pumps should be capable of rapid priming and repriming at the "lead pump on" elevation. Such self-priming and repriming should be accomplished automatically under design operating conditions. Suction piping should not exceed the size of the pump suction and should not exceed 7.6 m (25 ft) in total length. Priming lift at the "lead pump on" elevation should include a safety factor of at least 1.2 m (4 ft) from the maximum

[1] Consists of two spring loaded check valves operating in series and a diaphragm-activated, pressure differential relief valve, located between the check valves. Two shutoff valves with test cocks complete the device. Recommended for high health hazard risk where it would be impractical to have an air gap separation. Malfunctioning of this device is indicated by discharge of water from the relief port. The backflow preventers require periodic inspection, maintenance and induce high pressure loss. Cannot be installed below ground level and should be protected from freezing. Space for maintenance and testing should be provided.

allowable priming lift for the specific equipment at design operating conditions. The combined total of dynamic suction-lift at the "pump off" elevation and required *net positive suction head* (NPSH) at design operating conditions should not exceed 6.7 m (22 ft).

Vacuum-priming pumping stations should be equipped with dual vacuum pumps capable of automatically and completely removing air from the suction-lift pump. The vacuum pumps should be adequately protected from damage due to sewage. The combined total of dynamic suction-lift at the "pump off" elevation and required NPSH at design operating conditions should not exceed 6.7 m (22 ft).

Suction-lift pump stations using dynamic suction lifts exceeding the limits outlined above may be considered with the factory certification of pump performance and detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations need to include static suction-lift as measured from "lead pump off" elevation to centre line of pump suction, friction and other hydraulic losses of the suction piping, vapor pressure of the liquid, altitude correction, required NPSH and a safety factor of at least 1.8 m (6 ft).

7.3.2 Equipment, Wet Well Access and Valves Location

The pump equipment compartment should be above grade or offset and should be effectively isolated from the wet well to prevent a hazardous and corrosive sewer atmosphere from entering the equipment compartment. Wet well access should not be through the equipment compartment and should be at least 900 by 900 mm (36 by 36 in) hatch or larger. Gasketed replacement plates should be provided to cover the opening to the wet well for pump units removed for servicing. Valving should not be located in the wet well.

7.4 SUBMERSIBLE PUMPING STATIONS - SPECIAL CONSIDERATIONS

The designer should consider the applicable design recommendations provided in *Section 7.2 - Design* except as modified in this section.

7.4.1 Construction

Submersible pumps and motors should be designed specifically for raw sewage use, including totally submerged operation during a portion of each pumping cycle. An effective method to detect shaft seal failure or potential seal failure should be provided. Small pre-fabricated submersible pump stations are available with pumps and motors pre-designed into a single well.

7.4.2 Pump Removal

Submersible pumps should be readily removable and replaceable without personnel entering or dewatering the wet well, or disconnecting any piping in the wet well. Pump removal should include an engineered hoist.

7.4.3 Electrical Equipment

Electrical supply, control and alarm circuits should be designed to provide strain relief and to allow disconnection from outside the wet well. Terminals and connectors should be protected from corrosion by location outside the wet well or through use of watertight seals.

The motor control centre should be located outside the wet well, be readily accessible and be protected by a conduit seal or other appropriate measures meeting the requirements of the Ontario *Electrical Safety Code*, to prevent the atmosphere of the wet well from gaining access to the control centre. The seal should be so located that the motor may be removed and electrically disconnected without disturbing the seal. When such equipment is exposed to weather, it should meet the requirements of weatherproof equipment NEMA 3R or 4.

Pump motor power cords should be designed for flexibility and serviceability under conditions of extra hard usage and should meet the requirements of the Ontario *Electrical Safety Code* standards for flexible cords in sewage pump stations. Ground fault interruption protection should be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable. Power cord terminal fittings should be corrosion-resistant and constructed in a manner to prevent the entry of moisture into the cable, should be provided with strain relief appurtenances and should be designed to facilitate field connecting.

7.4.4 Valves

Required valves should be located in a separate valve chamber. Provisions should be made to remove or drain accumulated water from the valve chamber. The valve chamber may be dewatered to the wet well through a drain line with a gas and water tight valve. Check valves that are integral to the pump need not be located in a separate valve chamber provided that the valve can be removed from the wet well. Access should be provided in accordance with *Section 7.2 - Design*.

7.5 SCREW PUMP STATIONS – SPECIAL CONSIDERATIONS

The designer should consider the applicable design recommendations provided in *Section 7.2 - Design*. Screw pumps range in size, based on the diameter of the screw, from a minimum of 0.3 m (1 ft) to a maximum of 3.7 m (12 ft). The efficiency of the screw pump increases from its minimum capacity to its rated capacity based on the fluid level in the influent well. These units are well suited for variable speed capacity operation because the rate of discharge is controlled by the fluid level at the screw inlet. No variable-speed device is required for these pumps.

7.5.1 Covers

Covers or other means of excluding direct sunlight should be provided as necessary to eliminate adverse effects from temperature changes.

7.5.2 Pump Wells

A positive means of isolating individual screw pump wells should be provided.

7.5.3 Bearings

Submerged bearings should be lubricated by an automated system without pump well dewatering.

7.6 ALARM SYSTEMS

Alarm systems with a backup power source should be provided for all pumping stations. The alarm should be activated in cases of power failure, dry well sump and wet well high water levels, pump failure, unauthorized entry, or any other cause of pump station malfunction. Pumping station alarms including identification of the alarm condition should be transmitted to a municipal facility that is staffed 24-hours a day. If such a facility is not available and a 24-hour holding capacity is not provided, the alarm should be transmitted to municipal offices during normal working hours and to the home of the responsible person(s) in charge of the pumping station during off-duty hours. Audio-visual alarm systems may be acceptable in some cases in lieu of a transmitting system depending upon location, station holding capacity and inspection frequency.

7.7 STANDBY POWER AND EMERGENCY OPERATION

7.7.1 General

The designer should evaluate the need for standby power at a sewage pumping station for each specific location and should confirm this assessment with the ministry. The objective of emergency operation is to prevent (and in the case of combined sewer system to minimize) the discharge of raw or partially treated sewage to any waters and to protect public health by preventing back-up of sewage and potential discharge to basements, streets and other public and private property.

7.7.2 Emergency Pumping Capability

Emergency pumping capability is required unless on-system overflow prevention is provided by adequate storage capacity. Emergency pumping capability should be accomplished by provision of portable or in-place internal combustion engine equipment, which will generate electrical or mechanical energy, or by the provision of portable pumping equipment. For engine driven generating equipment, an automatic transfer switch should be provided to allow for bypass of unit for service. Such emergency standby systems should have sufficient capacity to start up and maintain the design capacity of the pumping station. Regardless of the type of emergency standby system provided, a portable pump connection to the forcemain with rapid connection capabilities and appropriate valving should be provided outside the dry well and wet well.

7.7.3 Emergency High Level Overflows

A controlled, high-level wet well overflow to supplement alarm systems and emergency power generation should be provided for use during possible periods of extensive power outages, mandatory power reductions, or uncontrollable emergency conditions. Where a high level overflow is utilized, consideration should also be given to the installation of storage/detention tanks, or basins, which should be made to drain to the pumping station wet well. Where such overflows may affect public water supplies or other critical water uses, the ministry should be contacted for the necessary treatment or storage requirements and in the case of combined sewer overflow the application of the ministry Procedure F-5-5 to the site-specific conditions.

7.7.4 Equipment Requirements

The following general requirements should apply to all internal combustion engines used to drive auxiliary pumps, service pumps through special drives, or electrical generating equipment:

- The engine should be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment should be capable of shutting down the engine and activating an alarm on site and as provided in *Section 7.6 - Alarm Systems*. Protective equipment should monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication;
- The engine should have adequate rated power to start and continuously operate under all connected loads;
- Reliability and ease of starting, especially during cold weather conditions, should be considered in the selection of the type of fuel;
- Underground fuel storage and piping facilities should be constructed in accordance with applicable provincial and federal regulations;
- The engine should be located above grade with adequate ventilation of fuel vapours and exhaust gases;
- All emergency equipment should be provided with instructions indicating the need for regular starting and running of such units at full loads; and
- Emergency equipment should be protected from damage at the restoration of regular electrical power.

7.7.5 Engine-Driven Pumping Equipment

Where permanently-installed or portable engine-driven pumps are used, the following requirements in addition to general requirements apply:

- Engine-driven pumps need to meet the design pumping requirements unless storage capacity is available for flows in excess of pump capacity. Pumps should be designed for anticipated operating conditions, including suction lift if applicable;
- The engine and pump need to be equipped for automatic start-up and operation of pumping equipment unless manual start-up and operation is justified. Provisions also need to be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm system needs to be provided to allow time for the detection of the pumping station failure and time to setup portable equipment; and
- Where part or all of the engine-driven pumping equipment is portable, sufficient storage capacity with alarm system needs to be provided to allow time for detection of pumping station failure and setup of portable equipment.

7.7.6 Engine-Driven Generating Equipment

Where permanently-installed or portable engine-driven generating equipment is required, the designer in addition to general design recommendations in *Section 7.7.4 - Equipment Requirements* should consider the following***:***

- Generating unit size should be adequate to provide power for pump motor starting current and for lighting, ventilation and other auxiliary equipment necessary for safety and proper operation of the pumping station;
- The operation of only one pump during periods of auxiliary power supply should be evaluated and justified. Such justification may be made on the basis of the design peak hourly flows relative to single-pump capacity, anticipated length of power outage and storage capacity; and
- Special sequencing controls should be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

Provisions needs to be made for automatic (i.e., automatic transfer switch (ATS)) and manual start-up and load transfer unless only manual start-up and operation is justified. The generator should be protected from operating conditions that would result in damage to equipment. Provisions should be considered to allow the engine to start and stabilize at operating speed before assuming the load. Where manual start-up and transfer is justified, storage capacity and alarm system needs to be provided to allow time for the detection of the pumping station failure and time to setup portable equipment. It is standard practice, when using diesel engines, to permit them to run for not less than 60 minutes to avoid sludging and other problems. For this reason the designer should provide a standby power system with manual start up or with automatic start up utilizing an adjustable delay timer to start up during momentary power failures which would prevent the diesel engine from unnecessarily running. Timers should also be provided to bring equipment on-line in such a way that the generators will not be overloaded by the starting current requirements of motors. Similar protection will be necessary to avoid overload of the normal electrical supply on resumption of power following a power failure.

Where portable generating equipment or manual transfer is provided, sufficient storage capacity with alarm system needs to be provided to allow time for detection of pump station failure and transportation and connection of generating equipment. The use of special electrical connections and double throw switches are recommended for connecting portable generating equipment.

7.8 OPERATIONS MANUAL

Sewage pumping stations and portable equipment should be supplied with a complete set of operational instructions, including emergency procedures, maintenance schedules, tools and such spare parts as may be necessary. Documentation to be kept at the pumping station should confirm the level at which flooding, in particular basement flooding, will occur. This level should be provided as an elevation and also co-related to levels in the pumping station wet well. The designer is referred to *Section 3.14 - Manuals & Training* for more details on operations and equipment manuals.

7.9 FORCEMAINS

7.9.1 Velocity and Diameter

At design pumping rates, a cleansing velocity of at least 0.6 m/s (2 ft/s) should be maintained or a range of between 0.6 and 1.1 m/s (2 to 3.6 ft/s); the maximum velocity should be limited to 3 m/s (10 ft/s). The minimum forcemain diameter for raw sewage should not be less than 100 mm (4 in), unless hydraulic computations are made. If the velocity in the forcemain is lower than 0.8 m/s (2.6 ft/s), hydraulic computations should be made to determine the pipe diameter if it is to be less than 100 mm (4 in), although the pipe diameter should not be less than 50 mm (2 in).

7.9.2 Air and Vacuum Relief Valves

An air relief valve should be placed at high points in the forcemain to prevent air locking. Vacuum relief valves may be necessary to relieve negative pressures on forcemains. The forcemain configuration and head conditions should be evaluated as to the need for and placement of vacuum relief valves.

7.9.3 Termination

The forcemain should enter the receiving manhole with a smooth flow transition to the gravity sewer system at a point not more than 0.3 m (1 ft) above the flow line. Corrosion protection should be provided where corrosive conditions are anticipated due to septicity or other causes. The forcemain length should be short to reduce dynamic headlosses and the production of odours and corrosive gases at initial and design flows, respectively.

7.9.4 Design Pressure

Pipe and joints should be equal to watermain strength materials suitable for design conditions. The forcemain, reaction blocking and station piping should be designed to withstand transient pressures and associated cyclic reversal of stresses that are expected with the cycling of sewage lift stations. The use of surge valves, surge tanks or other suitable means (e.g. slow closing check valves) to protect the forcemain against severe pressure changes should be evaluated. The designer should be aware of the reduced reliability of air and vacuum release valves and surge control valves when applied to sewage containing grease, grit and rags. The location of the pumping station or the forcemain should be such as to minimize intermediate high points that might result in column separation.

7.9.5 Special Considerations

Forcemain construction near streams or *water works* structures and at watermain crossings need to meet applicable provisions of <u>*Section 5.14 - Protection of Drinking Water Systems.*</u>

The designer should provide an external bypass and portable pump connection for small pumping stations.

7.9.6 Design Friction Losses

Friction losses through forcemains should be based on the Hazen-Williams formula or other acceptable methods. When the Hazen-Williams formula is used, the value for "C" should be 100 for unlined iron or steel pipe for design. For other smoother pipe materials (i.e., such as PVC, polyethylene, lined ductile iron) a higher "C" value not to exceed 120 may be considered.

When initially installed, forcemains will have a significantly higher "C" factor. The effect of the higher "C" factor should be considered in calculating maximum power requirements and duty cycle time to prevent damage to the motor. The effects of higher discharge rates on selected pumps and downstream facilities should also be considered. In evaluating existing systems for expansion, the C-factors should be determined by actual tests wherever possible.

7.9.7 Identification

Where forcemains are constructed of material which might cause the forcemain to be confused with potable watermains, the forcemain should be appropriately identified. The designer should consider designing the sewage forcemain with materials not used for watermains at the same location (e.g. PVC) to avoid cross-connections.

7.9.8 Leakage Testing

Leakage tests should be specified, including testing methods and leakage limits.

7.9.9 Maintenance Considerations

Isolation valves should be considered where forcemains connect into a common forcemain. Cleanouts at low points and chambers for pig launching and catching should be considered for any forcemain to facilitate maintenance.

7.9.10 Cover

Forcemains should be covered with sufficient earth or other insulation to prevent freezing. The required burial depth (i.e., frost penetration depth) varies across the province from approximately 1.2 m (4 ft) to greater than 3.0 m (10 ft). The designer should refer to *Chapter 6 - Challenging Conditions Affecting Servicing* for information on climatic factors impacting the design of sewage forcemains.

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CHAPTER 8

DESIGN CONSIDERATIONS FOR SEWAGE TREATMENT PLANTS

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DESIGN CONSIDERATIONS FOR SEWAGE TREATMENT PLANTS

This chapter describes design considerations as they relate to sewage treatment plants. Topics covered in this chapter include plant location, effluent quality requirements, design issues and details, outfalls, essential facilities and some general aspects of safety.

8.1 PLANT LOCATION

8.1.1 General

A new sewage treatment plant site or an expansion of an existing sewage treatment plant will be evaluated through the *Municipal Engineers Association's Municipal Class Environmental Assessment* (MEA's Municipal Class EA) process and will be documented in the Environmental Study Report (ESR). Some of the factors which should be taken into consideration in this evaluation include:

- Locations of drinking water sources, *surface water* intakes and *groundwater* wells;
- Adequacy of isolation from residential areas and land use surrounding plant site;
- Prevailing wind directions;
- Susceptibility of site to flooding;
- Suitability of soil conditions;
- Adequacy of site for future expansion and/or provision for additional treatment stages;
- Suitability of site with respect to access to receiving body of water or other means of treated sewage effluent disposal;
- Assimilation capacity of receiving water body;
- Acceptability of site with respect to sludge disposal/utilization options on site or access to areas off site; and
- Design *capacity* (see *Section 3.10 - Sewage Treatment Plant Capacity Rating*).

8.1.2 Flood Protection

The susceptibility of the site to flooding should be investigated and, if necessary, measures need to be taken to prevent flooding damage as may be directed by the local Conservation Authority or the Ontario Ministry of Natural Resources (MNR). The treatment plant structures, electrical and mechanical equipment should be protected from physical damage by the 100-year flood event. This requirement applies to new construction and to existing facilities undergoing expansion. Flood plain regulations of provincial and federal agencies and the municipal requirements related to flood plain protection need to be followed. The designer should also consider if high receiving water levels would impact the discharge of treated sewage effluent.

8.1.3 General Plant Layout

The general arrangement of the treatment plant within the site should take into account the subsurface conditions and natural grades to provide the necessary facilities at a minimum cost.

In the layout of the plant, the designer should orient the buildings to provide adequate allowances for future linear expansions of the various treatment stages and process units and orient the plant so that the best advantage can be taken of the prevailing wind and weather conditions. The building orientation can be used to minimize effects of odours, misting and freezing problems and energy usage (heating). The plant layout should also allow for the probability of snow drifting to minimize its effects on operations.

Within these constraints, the designer should work towards a plant layout where the various processing units are arranged in a logical progression to avoid the necessity for major pipelines or conduits to convey sewage, sludge, or chemicals from one module to the next and also to arrange the plant layout to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

Where site roadways are provided for truck access, the road design should be sufficient to withstand the largest anticipated delivery and disposal vehicles with due allowance for vehicle turning and forward exit from the site.

In order to avoid the dangers of high voltage lines crossing the site, a high voltage pole should be located at the property line. Depending on the distance from this pole to the control building, the step-down transformer could be located at the terminal pole. If the distance between the terminal pole and the building is excessive, the transformer should be located adjacent to the control building. The high voltage connections should be brought by underground cable to the pothead at the transformer. In this way, the primary and secondary terminals of the transformer are fully enclosed and no fence is required around the transformer.

Sewage treatment works sites should be adequately fenced and posted to prevent persons from gaining unauthorized access. The perimeters of open tanks or excavations should be adequately safeguarded. Gates and buildings should have locks. Consideration should be given to security enhancements such as alarm systems with access areas monitored using motion detectors and cameras.

8.1.4 Provisions for Future Expansion

In addition to the site size needed to physically accommodate future treatment plant expansions, it is necessary for the designer to include provisions to accommodate the future expansion and/or process changes.

On site sewage pumping stations should be designed such that their capacity can be increased and/or parallel facilities constructed without the need for major disruption of plant operation. The layout and sizing of channels and plant piping should be such that additional treatment units can be added in the future or increases in loading rates can be accommodated hydraulically. The location of buildings and tanks should allow for the location of the next stages of expansion. Buffer areas should be provided.

The need for septage and landfill leachate receiving facilities should be evaluated and appropriate space and provisions allocated (*Chapter 19 – Co-treatment of Septage and Landfill Leachate at Sewage Treatment Plants*).

Within buildings, space should be provided for the replacement of equipment with larger capacity units such as pumps, blowers, boilers and heat exchangers. Adequate working space should be provided around equipment and provision made for the removal of equipment for replacement, or major maintenance operations.

In sizing inlet and outlet sewers, the ultimate plant capacity should be considered. Provided that problems will not occur with excessive sedimentation in the sewers, these sewers should be sized for the ultimate condition. With diffused outfalls, satisfactory port velocities can often be obtained by blocking off ports which will not be required until subsequent expansion stages.

8.2 ESTABLISHMENT OF EFFLUENT QUALITY REQUIREMENTS

The *ministry* Guideline B-1, *Water Management - Policies, Guidelines and Provincial Water Quality Objectives* provides numerical and narrative ambient surface water quality criteria as provincial water quality objectives (PWQO). It then sets effluent requirements for sewage treatment plant discharges by introducing the concept of a mixing zone whereby PWQOs are to be met at the boundary of the mixing zone.

The Procedure B-1-5, *Deriving Receiving-Water Based, Point Source Effluent Requirements for Ontario Waters* provides the framework within which the ministry sets effluent requirements in terms of contaminant loadings and concentrations and incorporates these requirements into a *Certificate of Approval* (C of A) for a sewage treatment plant.

It is the responsibility of the proponent to conduct a site specific receiving water body assessment in order to determine the effluent requirements based on the assimilative capacity of the receiver. If the effluent requirements determined by the receiving water assessment are less stringent than those stipulated in federal or provincial effluent regulations or guidelines, then the most stringent of the latter will be imposed. The Ontario effluent guidelines are provided in Guideline F-5, "*Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters*". Guideline F-5 states:

"The normal level of treatment required for municipal and private sewage treatment works discharging to surface waters is secondary treatment or equivalent."

Secondary treatment is provided by biological processes (e.g. activated sludge process and its variations, fixed film processes) or physical-chemical processes producing an effluent quality of CBOD5 and TSS of 15 mg/L or better. In Ontario, the compliance limits for secondary treatment are typically set as not to exceed monthly average concentration of 25 mg/L for each of CBOD5 and TSS.

Sewage treatment lagoons producing an effluent quality of CBOD5 of 25 mg/L and TSS of 30 mg/L are considered as providing secondary equivalent treatment. In Ontario the compliance limits for lagoons are set as annual (or period of discharge) average concentrations of CBOD5 of 30 mg/L and TSS of 40 mg/L. The compliance limits for seasonal discharge lagoons with batch chemical dosing for phosphorus removal are usually CBOD5 of 25 mg/L and TSS of 25 mg/L.

Sewage treatment works that provide only primary settling of solids and the addition of chemicals to improve the removal of total phosphorus and/or solids are not considered as secondary treatment, or equivalent.

Sewage treatment works should also be able to produce final effluent quality that does not exceed monthly average total phosphorus (TP) concentration of 1 mg/L when phosphorus removal is required and a monthly geometric mean density of 200 *E. coli* organisms per 100 mL when disinfection is required.

Treatment beyond the norm of secondary or equivalent level for various *watersheds* may be necessary due to limited assimilation capacity and/or critical downstream uses being made of the receiving body of water. Many sewage treatment plants in Ontario are required to meet more stringent effluent quality requirements than associated with secondary treatment.

Some sewage treatment plants may also be required to produce a nitrified effluent, due to either concerns with un-ionized ammonia toxicity or nitrogen related oxygen demands in the receiving waters.

The receiving water-based effluent requirements should be confirmed by ministry regional staff. Once confirmed, the effluent quality requirements should serve as terms of reference for the design of the sewage treatment plant.

The most important decision that the designer should make, prior to designing sewage treatment works, is to set the design effluent quality objectives required to consistently achieve the compliance limits. It should be noted that the design objectives should reflect the specified time-averaged terms (weekly average, monthly average and annual average) used in defining effluent compliance limits.

The receiving water-based effluent requirements form the basis of the ministry technical review of the proposed sewage treatment works and are incorporated into the C of A as effluent compliance limits with appropriate effluent quality objectives in terms of concentrations and loadings.

Depending on the effluent requirements, there are a number of suitable alternative sewage treatment processes that can be considered. Table 8-1 lists some of the treatment processes and the expected effluent quality produced by well designed and operated plants for treating municipal sewage.

Other factors, in addition to expected effluent quality which will affect the choice of treatment processes are:

- Ultimate sludge disposal options;
- Available land area;
- Operator skills;
- Soil conditions;
- Need for retention of treated sewage during periods of the year where receiving streams experience insufficient flows or where downstream recreational water uses make summer effluent discharges undesirable;
- Future loads such as hauled septage and/or landfill leachate handling; and
- Capital and operation and maintenance (O&M) costs.

Before deciding upon the sewage and sludge treatment processes, the designer should evaluate the alternatives available, in terms of treatment capability and overall capital and O&M costs, to ensure that the most appropriate treatment system is selected.

Table 8-1 – Sewage Treatment Processes with Typical Effluent Quality

Process	Effluent Parameters (mg/L)			
	$CBOD_5$	TSS	TP	TAN
CONVENTIONAL ACTIVATED SLUDGE				
- Without P Removal	15	15	3.5	15 - 20
- With P Removal	15	15	<1.0	15 - 20
- With P Removal and Filtration	5	5	0.3	15 - 20
- With Nitrification and P Removal	15	15	<1.0	3
CONTACT STABILIZATION				
- Without P Removal	20	20	3.5	15 - 20
- With P Removal	20	20	<1.0	15 - 20
EXTENDED AERATION				
- Without P Removal	15	15	3.5	3
- With P Removal	15	15	<1.0	3
- With P Removal and Filtration	5	5	0.3	3
BIOLOGICAL NUTRIENT REMOVAL				
- With P and Nitrogen Removal	15	15	<1.0	3 (TN< 6)
FIXED FILM PROCESSES (RBC, TRICKLING FILTER)				
- Without P Removal	15	20	4.0	15 - 20
- With P Removal	15	20	<1.0	15 - 20
- With P Removal and Filtration	10	5	0.3	15 - 20
- With Nitrification and P Removal	15	20	<1.0	3.0
MEMBRANE BIOREACTOR				
- Without P Removal	2	1	3.0	15 - 20
- With P Removal	2	1	0.1	15 - 20
- With Nitrification and P Removal	2	1	0.1	0.3
CONTINUOUS DISCHARGE LAGOON				
- Without P Removal	25	30	6.0	-
- With P Removal	25	30	<1.0	-
SEASONAL DISCHARGE LAGOON				
- Without P Removal	25	30	6.0	-
- With P Removal by Batch Chemical Dosage	15	20	<1.0	-
- With P Removal by Continuous Chemical Dosage	25	30	<1.0	-
AERATED FACULTATIVE LAGOON				
- Without P Removal with 4-5d Retention Time	60	100	6.0	-

Notes:

1. The above values are based on raw sewage with BOD_5 = 150-200 mg/L, Soluble BOD_5 = 50% of BOD_5, TSS = 150-200 mg/L, TP = 6-8 mg/L, TKN = 30-40 mg/L, TAN = 20-25 mg/L.
2. TAN (total ammonia nitrogen) concentrations may be lower during warm weather conditions if nitrification occurs.

8.3 DEFINITIONS OF TERMS

8.3.1 Biochemical Oxygen Demand Test

The standard 5-day Biochemical Oxygen Demand test measures the oxygen utilized during a 5-day period for the biochemical degradation of organic material and to oxidize inorganic material such as sulphides and ferrous iron. Significant nitrogenous oxygen demand can be exerted during the testing when a sufficient population of nitrifying bacteria (nitrifiers) and quantity of ammonia and/or nitrites are present in the test samples with low organic content (such as in many secondary effluents). In such cases, an inhibitor may be used during the testing to suppress the nitrogenous oxygen demand. The oxygen demand exerted by the oxidation of inorganic material in sewage is usually not significant.

If the nitrogenous oxygen demand is suppressed by using an inhibitor, the test results are referred to as Carbonaceous Biochemical Oxygen Demand ($CBOD_5$). If both $CBOD_5$ and nitrogenous oxygen demands are measured (without using an inhibitor), the resulting oxygen demand is simply referred to as Biochemical Oxygen Demand (BOD_5) which is also known as Total Biochemical Oxygen Demand ($TBOD_5$).

8.3.1.1 $CBOD_5$

$CBOD_5$ should be used for the assessment of secondary (or higher) sewage treatment works performance and as an indicator of their effluent quality.

Effluents from sewage treatment plants (STPs) exhibiting partial nitrification (with both nitrifiers and ammonia present) may have higher BOD_5 values than those with no nitrification (with no nitrifiers present) or complete nitrification (with no ammonia present). Since most factors that are conducive to improved effluent quality from secondary STPs are also conducive to nitrification, the effluent BOD_5 values can erroneously indicate poorer quality when, in fact, both the effluent quality and the plant performance are indeed good. This is often the case during warm weather periods or in newer treatment plants that are organically under loaded.

8.3.1.2 BOD_5

The designer should use BOD_5 for the assessment of raw sewage and primary effluents in estimating design parameters such as organic loadings and process air requirements of the secondary treatment process.

Although both BOD_5 and $CBOD_5$ are expected to be the same in raw sewage and primary effluents, there are cases where $CBOD_5$ has consistently underestimated the organic strength of these sewage streams.

8.3.2 Sewage Treatment Plant Design Capacity

The sewage treatment plant design capacity should be such that the treated effluent would continuously meet the established quality criteria in terms of concentrations and loadings during the design period (*Section 8.2 - Establishment of Effluent Quality Requirements*). For plants serving combined sewers subject to excessive wet weather flows or overflow detention pump-back flows, the design maximum day flows that the plant is to treat on a sustained basis should be taken into consideration. (*Chapter 21 – Control and Treatment of Combined Sewer Overflows*)

8.3.3 Sewage Treatment Plant Rated Capacity

Rated Capacity of a sewage treatment plant for Municipal Engineers Association Class Environmental Assessment (MEA Class EA) requirements generally means the design average daily flow for the limiting process stage (e.g., secondary treatment stage). For more information on details for establishing STP rated capacity and STP re-rating refer to *Section 3.10 - Sewage Treatment Plant Capacity Rating.*

8.3.4 Combined Sewer System

A combined sewer system is a sewage collection system which conveys sanitary sewage (domestic, commercial and industrial wastewaters) and stormwater runoff through a single-pipe system to a sewage treatment plant. Combined sewer systems which have been partially separated and in which roof leaders and/or foundation drains contribute stormwater inflow to the sewer system conveying sanitary flows are still defined as combined sewer systems in the ministry Procedure F-5-5, "*Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems*".

8.3.5 Sanitary Sewer System

A sanitary sewer system is a separate sewer system which conveys sanitary sewage (domestic, commercial and industrial wastewaters), infiltrated groundwater and limited amounts of stormwater where an adjoining separate storm sewer system exists as the primary collection system to receive stormwater flows from catch basins and other sources of stormwater.

8.3.6 Dry Weather Flow

Sewage flow resulting from sanitary wastewater (combined input of domestic, commercial and industrial flows) and infiltration and inflows from sewer joints and service connections, during periods with an absence of rainfall or snowmelt.

8.3.7 Wet Weather Flow

Sewage flow resulting from sanitary wastewater (combined input of domestic, commercial and industrial flows) infiltration and inflows from sewer joints and service connections, during periods of rainfall or snowmelt; or stormwater generated by either rainfall or snowmelt that enters the sanitary sewer system or combined sewer system.

8.3.8 Average Daily Flow

The average daily flow is the average of the daily volumes to be received in a calendar year expressed as a volume per unit time. The average daily flow for *sewage works* having critical seasonal high hydraulic loading periods (e.g., recreational areas, campuses and industrial facilities) should be based on the average of the daily volumes to be received during the seasonal period.

8.3.9 Peak Daily Flow

The peak daily flow is the largest volume of flow to be received during a one day period expressed as a volume per unit time. This flow is also referred to as maximum daily flow or maximum day flow[1].

8.3.10 Peak Hourly Flow

The peak hourly flow is the largest volume of flow to be received during a one-hour period expressed as a volume per unit time. This is also referred to as maximum hourly flow or maximum hour flow.

8.3.11 Peak Instantaneous Flow

The peak instantaneous flow is the instantaneous maximum flow rate as measured by a metering device.

8.3.12 Minimum Daily Flow

The minimum daily flow is the smallest volume of flow to be received during a continuous one-day period expressed as a volume per unit time. This is also referred to as minimum day flow. Initial low flow conditions in new sewage works should be evaluated in the design to minimize operational problems with freezing, septicity, flow measurements and solids dropout.

[1] The designer should be judicious in application of this number.

8.3.13 Design Flows

The designer should select appropriate flow rates for the design of specific process units in sewage works. These flows would be designated as design flows (e.g., design average daily flow, design peak daily flow and design peak hourly flow). For more information on recommended design flows for various STP components refer to Table 8-2.

8.3.14 Bypass

Bypassing of any treatment processes within a sewage treatment plant with the associated sewage flows being returned to the sewage treatment plant flow and discharging to the environment through the final effluent outfall of the sewage treatment plant.

8.3.15 Overflows

- Combined Sewer Overflow means a discharge to the environment from a combined sewer system;
- Sanitary Sewer Overflow means a discharge to the environment from sanitary sewer system; and
- Sewage Treatment Plant Overflow means a discharge to the environment from a sewage treatment works at a location other than the final effluent outfall or downstream of the sampling point in the final effluent outfall.

8.3.16 Emergency

A condition that if not mitigated, could result in personal injury or loss of life, structural damage to the sewage works, basement flooding or other health hazard.

8.3.17 Unavoidable Condition

A condition beyond the reasonable foresight or control of the owner and operator of the works and includes exceptional acts of nature, third party actions (e.g., vandalism), or structural, mechanical or electrical failure.

8.4 BASIS OF PROCESS SELECTION

The selection of an appropriate process(es) for a sewage treatment plant is unique to each site and conditions. Subsequent chapters in these design guidelines outline options for each process and associated advantages and disadvantages for each. Many issues need to be considered during process selection, such as:

- Influent characteristics;
- Influent flows, including average, minimum and peak flows;

- Effluent requirements (*Section 8.2 - Establishment of Effluent Quality Requirements*);
- Compatibility with other processes;
- Local conditions;
- Local resources;
- End use for byproducts; and
- Economics.

8.4.1 Value Engineering Approach

The procedure to develop alternative options for a sewage treatment plant should involve a review of the needs and limitations through a comprehensive evaluation process. One method could include a workshop or value engineering approach, followed by detailed characterization of the needs and abilities of each option and an overall weighted evaluation to determine the preferred alternative.

The value engineering (VE) approach is an intensive workshop during which the project design is analyzed for optimization of cost, energy, operation and maintenance.

The VE Job Plan is important because it provides an organized approach to identifying high initial capital, energy and life-cycle costs. The functional requirements needed to operate and maintain the facilities are analyzed to ensure performance. Where the essential functions are not being furnished by the design, there is a lack of "value". The VE team identifies alternative approaches that will provide the needed value. Portions of the project not functionally required or carrying major parts of project costs are likely targets for team evaluation. Developing recommendations to reduce these high cost areas is an important aspect of the workshop.

The workshop is conducted in six phases in this specific order:

- Orientation Phase;
- Information Phase;
- Creative Phase;
- Judgment Phase;
- Development Phase; and
- Presentation Phase.

An alternatives development workshop (facilitated workshop) can be used to identify treatment process options that require further evaluation to determine the preferred options. Once the alternative options have been conceptually developed, an evaluation matrix approach can be used to review the alternatives identified.

8.4.2 New and Innovative Technologies

The designer should refer to *Section 3.9 - Technology Development* for details.

8.5 MAJOR DESIGN CRITERIA

The following major design criteria should be applied to the design of new sewage treatment plants, plant expansions or upgrades:

8.5.1 Sanitary Sewer Systems

- Sanitary sewer systems should be designed with the objective of conveying all the flows to be treated at the sewage treatment plant. Any overflows within the sanitary sewer systems and overflows/bypasses at treatment works should be designed for emergency and unavoidable conditions only;
- The biological treatment processes at the treatment plant should be designed to meet effluent quality requirements over a wide range of flows including design minimum, average and peak flows for the projected design period. Ontario experience for medium to large sewage treatment plants demonstrates design peak flows in the range of 2 to 3 times the design average daily flows;
- Where treatment process units need to accommodate any peak flows the design criteria should be the design peak hourly flows unless indicated otherwise;
- In cases where peak hourly flow to average daily flow ratio is exceptionally high, which may result in washout of the biomass necessary for treatment, or in unfavorable operating conditions at minimum flow rate (e.g. small STPs), judicious design peak hourly flows should be established by the designer to be best suited for the operation of the treatment process units. Peak flow mitigation efforts such as equalization techniques or standby units to be brought on-line when necessary (i.e., during wet weather conditions), may form part of the decision making process by the designer;
- Any bypasses within a sewage treatment plant should be reintroduced into the outfall prior to final effluent sampling; and
- Every effort should be made to treat all flows received at the STP within the sewage treatment capability of the individual unit processes using measures to provide the highest possible treatment. This could be facilitated by:
  - Reducing infiltration and inflow (I/I) to the collection system;
  - Maximizing the storage capacity of the collection system for equalization; and
  - Providing off-line storage in the collection system.

8.5.2 Combined Sewer Systems

Combined sewer systems should comply with ministry Procedure F-5-5:

- For an average year, only 10% of the wet weather flow during the seven-month period of concern that are above the dry weather flows from combined sewer systems may be allowed to overflow. During wet weather, the minimum level of treatment required for flows above the dry weather flows from combined sewer system is primary treatment;
- In cases where one area is served by combined (and or partially separated) sewers and the other area in the same sewer shed is served by sanitary sewers, Procedure F-5-5 applies only to the flows from the area served by the combined sewer systems; and
- For sewage collection systems consisting of both sanitary sewers and combined sewers, the design should be based on a distinction between all sanitary sewage flows from the entire system which should be conveyed to and treated at STP and combined sewer wet weather flows that are above the dry weather flows which are subject to Procedure F-5-5 requirements.

8.5.3 Design Period

Factors which will have an influence on the design period of sewage treatment plants include the following:

- Population growth rates;
- Sewershed boundaries;
- Heavy water use industries;
- Inflation and financing interest rates;
- Ease of expansion of facilities; and
- Time requirements for design and construction of any expansion.

Wherever possible, sewage treatment plants should be designed for the flows expected to be received during the next 20 years, under normal growth conditions. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used.

8.5.4 Sewage Flows

Wherever there are existing sewers and/or existing sewage treatment plants, the flow rates and sewage characteristics should be determined using real data, in both wet and dry weather conditions. Data collected should be analyzed to estimate the following:

- Average and peak flows of sewage generated within buildings serviced by the sewer system exclusive of any extraneous flows; and
- Average and peak infiltration and inflow for the design year.

During investigations to determine peak extraneous flows, it may be found that such flows are excessive and that measures should be taken to reduce these flows rather than provide flow equalization and/or treatment facilities to accommodate such excessive flows. It is often difficult to determine when measures to reduce infiltration will be cost effective. North American experience has indicated that if infiltration, based upon the highest weekly average within a 12-month period, is less than 0.14 L/(mm·d)/m (litres per millimetre of pipe diameter per day per linear metre of sewer length) [(0.28 US gal/(in·d)/ft)] rehabilitation of the sewer system will not be economical. The issue of extraneous flows and mitigating measures should be addressed during the MEA Municipal Class EA process.

Where it is not possible to base estimates of sewage flows and characteristics upon actual field measurements, the flow records and sewage characteristics of similar serviced communities may provide data upon which estimates can be based.

In estimating sewage flows, it is recommended that no less than 225 L/(cap·d) [59 US gal/(cap·d)] be used for average domestic sewage flows, exclusive of extraneous flows.

To estimate peak sewage flows, the average domestic flow rate (sewage flows from residential sources) should be multiplied by the Harmon factor, then the peak extraneous flows should be added. Industrial and commercial sewage flow rates should be calculated separately and added to the above sewage flow rates. (*Section 5.5.2 - Design Sewage Flows*)

8.5.5 Organic Loadings

Organic loadings for sewage treatment plant design should be based on actual data for the facility or a mass balance of the expected contributors. The effects of other high strength liquid wastes, such as septage and leachate, which may be accepted at the plant, should be evaluated and appropriate facilities should be included in the design (*Chapter 21 – Control and Treatment of Combined Sewer Overflows*).

Where it is found that sewage characteristics vary significantly over the year due to excessive infiltration/inflow, population variations and/ or seasonal changes in industrial or commercial operations, estimates should be made of the expected average, maximum and minimum BOD_5 and suspended solids concentrations in the sewage for each month of the year. If nitrification is required, short and long term variations in TAN (total ammonia nitrogen) and TKN (Total Kjeldahl Nitrogen) concentrations should also be estimated.

As part of the assessment of the influent sewage characteristics, the designer should consider the restrictions established by the local municipal sewer use bylaw.

The shock effects of high concentrations and diurnal peaks for short periods of time on the treatment process, particularly for small treatment plants, should be considered.

Typical organic loading rates for domestic sewage are 75 and 90 g/(cap·d) [2.6 and 3.2 oz/(cap·d)] for BOD_5 and SS, respectively.

8.5.6 Bypasses and Overflows

8.5.6.1 General

If sewage entering the treatment plant is to be pumped into the treatment units, an emergency overflow for the pumping station should be provided. This overflow should be routed through the disinfection facilities and plant outfall sewer. If this is not possible, provision should be made for separate disinfection of such overflows.

The overflow elevation and the method of activation should ensure that the maximum feasible storage of the sewage collection system and wet well would be utilized before the controlled overflow takes place. The overflow facilities should at least be alarmed and equipped to indicate frequency and duration of overflows and provided with facilities to permit flow measurement. Automatic flow measurement and recording systems may be required in certain cases where requirements dictate.

To allow maintenance operations to be carried out, each unit process within the treatment plant should be provided with bypass capability around the unit.

Where two or more similar treatment units are considered and one unit is out of operation for repairs, the remaining units should be capable of treating the design peak sewage flow rates or be provided with bypass capacity equal to the excess hydraulic flow of the operating units.

Bypass systems should also be constructed so that each unit process can be separately bypassed (i.e., no need to bypass more unit processes than necessary).

All flows bypassing secondary and/or tertiary treatment processes should be measured.

If a bypass for the chlorine contact chamber or other disinfection process (e.g. UV) is needed, provisions may be necessary for emergency disinfection of flows in the bypass channel.

8.5.6.2 Bypasses at Sewage Treatment Plants Serving Sanitary Sewers

Bypassing of treatment processes should not occur during dry-weather flow conditions, or during wet-weather flows equal to or lower than the design peak flows of the treatment works associated with sanitary sewer systems. It is recognized that the ability to bypass some processes should be incorporated into the design of the treatment works to minimize process washouts that may cause prolonged episodes of poor treatment performance.

Bypassing of one or more treatment process occurs when the volume of flow exceeds the maximum capacity of the treatment works associated with sanitary sewer systems.

The sanitary sewer systems while not designed to receive the bulk of stormwater flows, may experience increased flows during wet weather conditions as a result of increased infiltration and inflow through pipe joints and other sources of extraneous flows, including service connections, as well as direct and indirect sources of stormwater. These additional flows may, under certain circumstances, cause the capacity of the treatment plant to be exceeded.

Other reasons which can cause the capacity to be exceeded include, but are not limited to, structural, mechanical, or electrical failure, exceptional acts of nature and third party actions such as vandalism.

8.5.6.3 Overflows at Sewage Treatment Plants Serving Sanitary Sewers

Overflows from sewage treatment works associated with sanitary sewer systems should not occur. Overflow facilities may be incorporated into the design of STPs to prevent or mitigate an emergency caused by unavoidable conditions.

8.5.6.4 Bypasses and Overflows at Sewage Works Serving Combined Sewers

Bypasses and overflows at sewage works serving combined sewers are addressed in ministry Procedure F-5-5, *Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems*.

8.5.7 Pumping of Sewage

Raw sewage and any intermediate pumping stations within sewage treatment plants should be capable of conveying the peak sewage flow rates to the downstream treatment units. Pumping equipment should also be designed so that downstream treatment units receive a steady flow (with minimal flow rate variations). To achieve this, the pumping system can be provided with variable capacity, or multiple fixed capacity pumps, so that pump discharge rates will closely match the sewage inflow rate.

The designer should refer to *Chapter 7 - Pumping Stations* for the recommended design criteria.

8.5.8 Distribution of Flows and Organic Loadings

There will invariably be situations within STPs where flow splitting is necessary. To ensure that the organic load splits in the same proportion as the flows, the suspended solids should be homogeneously dispersed throughout the liquid and the relative momentum of all particles should be approximately equal at the point of diversion. Some turbulence is therefore desirable before each point of diversion. Channel or pipe bends upstream of flow division resulting in uneven solids distribution should be avoided. The following methods can be used to provide homogeneity:

- Mechanical mixers;
- Diffused aeration;
- Bottom entrance into splitting box;
- Bar racks or posts in channels;
- Hydraulic jumps; and
- Straight section of conduit 6 to 8 diameters (or channel width) upstream of point of diversion.

Flow division control facilities should be provided as necessary to ensure organic and hydraulic loading control to plant process units and should be designed for easy operator access, adjustment, observation and maintenance. The use of upflow division boxes equipped with adjustable sharp-crested weirs or similar devices is recommended. The use of valves for flow splitting is not acceptable. Appropriate flow measurement facilities should be incorporated in the flow division control design. The designer is referred to *Section 3.13 - Hydraulics* for more details.

8.5.9 Plant Hydraulic Gradient

The hydraulic gradient of all gravity flow and pumped sewage streams within the sewage treatment plant, including bypass channels, should be prepared to ensure that adequate provision has been made for all head losses. In calculating the hydraulic gradient, changes in head caused by all factors should be considered, including:

- Head losses due to channel and pipe wall friction;
- Head losses due to sudden enlargement or contraction in flow cross section;
- Head losses due to sudden changes in direction such as at bends, elbows, wyes and tees;
- Head losses due to sudden changes in slope or drops;

- Head losses due to obstructions in conduit;
- Head required to allow flow over weirs, through flumes, orifices and other measuring, controlling or flow division devices;
- Head losses caused by flow through comminutors, bar screens, tanks, filters and other treatment units;
- Head losses caused by air entrainment or air binding;
- Head losses incurred due to flow splitting along the side of a channel;
- Head increases caused by pumping;
- Head allowances for expansion requirements and/or process changes; and
- Head allowances due to maximum water levels in receiving waters.

Designers are cautioned to consider the consequences of excessive or inadequate allowances for head losses through sewage treatment plants. If pumping is required, excessive head loss allowances result in energy wastage. If inadequate head loss allowances are made, operation will be difficult and plant expansion would be more costly.

8.5.10 Sludge Pumping

The flow characteristics of sludge will vary according to the types and concentrations of organic solids and added chemicals. Some sludge flow characteristics will be similar to those of water while others may have pseudo plastic flow characteristics. The friction losses associated with sludge pumping applications vary greatly. Dilute sludge has losses similar to those experienced with clean water, whereas, thickened sludge has greater losses up to 15 times those of clean water. With sludge pumping, velocities of 0.9 to l.5 m/s (3.0 to 4.9 ft/s) should be developed. For heavier sludge and grease, velocities of 1.5 to 2.4 m/s (4.9 to 7.9 ft/s) are needed. To avoid blockages, a minimum line size of 150 mm (6 in) should be used. Mixing primary scum and grease with sludge lines may result in excessive headlosses and plugging and should be avoided.

8.5.11 Design Basis for Various Plant Components

The plant design should provide the necessary flexibility to perform satisfactorily within the expected range of influent sewage characteristics and flows.

All components of sewage treatment plants should be hydraulically capable of handling the anticipated peak sewage flow rates without overtopping channels and/or tanks. From a process point-of-view, design of the various process units or components of sewage treatment plants should be based upon the hydraulic, organic and inorganic loading rates shown in Table 8-2.

Table 8-2 – Unit Process Design Basis

Area/Process	Design Basis
Sewage Pumping Stations	Design Peak Instantaneous Flow
Screening	Design Peak Instantaneous Flow
Grit Removal	Design Peak Hourly Flow, Peak Hourly Grit Loading
Primary Sedimentation	Design Peak Daily Flow
Aeration (without nitrification)	Average Daily BOD_5 Loading (based on Design Average Daily Flow)
Aeration (with nitrification)	Average Daily BOD_5 loading (based on Design Average Daily Flow), Peak Daily TKN Loading (based on Design Peak Daily Flow)
Secondary Sedimentation	Design Peak Hourly Flow, Peak Daily Solids Loading
Sludge Return	For activated sludge processes, 50 to 200 % of Design Average Daily Flow
Disinfection	Design Peak Hourly Flow
Effluent Filtration	Design Peak Hourly Flow
Outfall Sewer	Design Peak Instantaneous Flow
Sludge Treatment (digestion and dewatering.)	Maximum Monthly Mass Loading and Flow Rates

8.5.12 Flow Equalization

Sewage flows can be fully or partially equalized. Full equalization may result in a small reduction in construction costs over variable flow design and in addition can result in reduced energy costs and improved treatment efficiency. Alternatively, partial equalization of sewage flows is an option that has decreased benefits with slight savings in construction costs.

8.5.13 Conduits

All piping and channels should be designed to carry the design peak instantaneous flows. The incoming sewer should be designed for unrestricted flow. Channels should be designed to convey the initial and ultimate range of expected flows. To avoid solids buildup, the following scouring velocities should be developed in normally used channels at least once per day:

- Sewage Containing Grit - 0.9 m/s (3.0 ft/s); and
- Floc Suspensions - 0.45 to 0.60 m/s (1.5 to 2.0 ft/s).

Where the above velocities cannot be obtained, channels should be aerated to prevent solids deposition. Bottom corners of the channels should be filleted. Conduits should be designed to avoid creation of pockets and corners where solids can accumulate.

Suitable gates or valves should be placed in channels to seal off unused sections which might accumulate solids. The use of shear gates, stop plates or stop planks may be considered where they can be used in place of gate valves or sluice gates. Non-corrodible materials should be used for these control gates.

8.5.14 Flow Measurement

Flow measurement facilities should be provided for the following flows:

- Plant influent or effluent flow. If influent flow is significantly different from effluent flow, both should be measured. This would apply for installations such as lagoons, sequencing batch reactors and plants with excess flow storage or flow equalization;
- Excess flow treatment facility discharges (e.g. equalization tank effluents);
- Overflows and bypasses to be monitored as required by the ministry; and
- Other flows such as return activated sludge, waste activated sludge and recycle flows required for plant operational control.

Indicating, totalizing and recording flow measurement devices should be provided for all mechanical plants. Flow measurement facilities for lagoon systems should not be less than elapsed time meters used in conjunction with pumping rate tests or else should be calibrated weirs. All flow measurement equipment should be sized to function effectively over the full range of flows expected and should be protected against freezing.

Flow measurement equipment including approach and discharge conduit configuration and critical control elevations should be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Conditions that need to be avoided include turbulence, eddy currents and air entrainment that upset the normal hydraulic conditions that are necessary for accurate flow measurement.

8.5.15 Sampling Equipment

Effluent composite sampling equipment should be provided at all mechanical plants with a design average daily flow of 380 m^3/d (0.1 mUSgd) or greater and at other facilities where monitoring effluent quality criteria parameters are necessary. Composite sampling equipment should also be provided as needed for influent sampling and for monitoring of plant performance. The influent sampling point should be located prior to any process return flows.

8.5.16 Component Backup Requirements

The designer should refer to *Section 3.8 - Reliability and Redundancy* for details on how to carry out the reliability and redundancy analysis of specific process trains, units or equipment.

The components of sewage treatment plants should be designed in such a way that equipment breakdown and normal maintenance operations can be accommodated with a minimal deterioration of effluent quality.

To achieve this, critical treatment processes should be provided with multiple units so that with the largest unit out of operation, the hydraulic capacity of the remaining units should be sufficient to handle the appropriate design sewage flows outlined in Table 8-2**.** There should also be sufficient flexibility in operation so that the normal flow into a unit that is out of operation can be distributed to all of the remaining units. It should be possible to distribute the flow to all of the units in the treatment process downstream of the affected process. In addition, where feasible, it should be possible to operate the sections of treatment plants as completely separate process trains to allow full-scale loading tests to be carried out.

With some processes such as mechanical screening or comminution, the back-up facility can be provided with a less sophisticated unit such as a manually cleaned screen. The designer should provide a bypass around a manual screen to avoid flooding.

Sewage and sludge pumping systems should always be provided with a back up pump of equal capacity to the largest duty pump. In certain cases, particularly with sludge pumps, one duty pump may serve as a back up for more than one set of pumps (e.g. a raw sludge pump could back up a sludge transfer pump). Standby capacity for sludge return pumps may be determined on a case-by-case basis.

Depending upon the size of the sewage treatment plant and the sensitivity of the receiving waters, some unit processes may not require duplication. For instance, if the equivalent of primary treatment would be satisfactory under emergency conditions, one aeration basin may be sufficient.

Aeration systems will require facilities to permit continuous operation, or minimal disruption, in the event of equipment failure. The following factors should be considered when designing the back-up requirements for aeration systems:

- Effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for example, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);
- Time required to perform the necessary repair and maintenance operations;

- The general availability of spare parts and the time required for delivery and installation. Preferably the vital spare parts should be stored on site;
- Means other than duplicate equipment to provide the necessary capacity in the event of a breakdown (e.g. using oversized mechanical aerators with adjustable weirs to control power draw and oxygenation capacity, or using two-speed mechanical aerators); and
- Diffused aeration systems require a standby blower, however mechanical aeration systems may not require standby units, depending upon the number of duty units and availability of replacement parts.

Chemical feed equipment for phosphorus removal and disinfection should be provided in multiple units so that the chemical requirements can be supplied with one unit out of service.

For sludge digestion facilities at small plants, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumping and mixing so that one stage can be serviced while the other stage receives the pumped raw sludge. When such an approach is proposed, the designer should outline the alternate methods of treatment and disposal that could be used during periods of equipment breakdown. For larger treatment plants, the provision of multiple primary and secondary digestion units can usually be economically justified.

Depending upon the receiving stream sensitivity, type of filtration equipment and the maintenance requirements of the filter units, provision of multiple effluent filtration units is often necessary.

For sludge handling and dewatering equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown.

8.5.17 Unit Bypasses

Properly located and arranged bypass structures and piping should be provided so that each unit of the STP can be removed from service independently. The bypass design should facilitate plant operation during unit maintenance and emergency repair so as to minimize deterioration of effluent quality and ensure rapid process recovery upon return to normal operational mode.

Treatment during bypassing may be accomplished through the use of duplicate or multiple treatment units in any stage if the design peak instantaneous flow can be handled hydraulically with the largest unit out of service.

The actuation of all bypasses should require manual action by operating personnel. All power-actuated bypasses should be designed to permit manual operation in the event of power failure. They should also be designed so that the valve will fail as is, upon failure of the power operator.

A fixed high water level bypass overflow to the bypass channel should be provided in addition to a manually or power actuated bypass. The designer should refer to *Section 8.5.6 - Bypasses and Overflows* for additional information.

8.5.18 Unit Dewatering, Flotation Protection and Plugging

Means such as drains or sumps should be provided to completely dewater each unit and to discharge to an appropriate point in the treatment process. Due consideration should be given to the possible need for hydrostatic pressure relief devices to prevent flotation of structures. Pipes subject to plugging should be provided with means for mechanical cleaning or flushing.

8.5.19 Construction Materials

Materials should be selected that are appropriate under conditions of exposure to hydrogen sulphide and other corrosive gases, greases, oil and other constituents frequently present in sewage. This is particularly important in the selection of metals and paints. Contact between dissimilar materials should be avoided or other provisions made to minimize galvanic action.

8.5.20 Installation of Mechanical Equipment

The specifications should be so written that the installation and initial operation of major items of mechanical equipment will be inspected and approved by a representative of the manufacturer.

8.5.21 Operating Equipment

A complete outfit of tools, accessories and spare parts necessary for the plant operator's use should be provided.

Readily accessible storage space and workbench facilities should be provided. Consideration should be given to provision of a garage for large equipment storage, maintenance and repair.

8.5.22 Erosion Control During Construction

Effective site erosion control should be designed for implementation during construction.

8.5.23 Grading and Landscaping

Upon completion of the plant, the ground should be graded and then sodded or seeded. All-weather walkways should be provided for access to all units. Steep slopes should be avoided to prevent erosion. Surface water should not be permitted to drain into any unit. Particular care should be taken to protect sludge beds and intermittent sand filters from stormwater runoff. Provision should be made for landscaping, especially when a plant needs to be located near residential areas.

8.6 PLANT OUTFALLS

The proper site and design of the plant outfall structure is important in minimizing the impact on receiving water quality. In many cases it may be a controlling factor in ensuring protection of nearby water supplies, recreational beaches or fisheries habitats.

Outfalls should be designed and located so as to obtain the greatest possible dilution of the plant effluent during periods of greatest susceptibility of nearby water uses to adverse impacts.

Dilution is a product of initial mixing of the effluent with surrounding water and subsequent dispersion due to water movement.

Entrainment of ambient lake or stream water into the effluent is generally enhanced by extending the outfall away from the shore into deeper water and often by incorporating a multi-port diffuser to spread the discharge over a larger area and to increase turbulent mixing. Similarly, dispersion is aided by maximizing the separation of the discharged plume from boundary effects of the shoreline and lake or streambed.

Reference should be made to the implementation procedure for defining mixing zones contained in the most recent version of the ministry Guideline B-1, *Water Management - Policies, Guidelines and Provincial Water Quality Objectives.*

Dispersion predictions require knowledge of effluent concentration, discharge rates, effluent buoyancy, jet velocity, ambient current velocity, depth of water over the outfall, ambient thermal regime (vertical temperature profile) and background water quality.

For all extended outfalls, outfall capacity should be sufficient to handle not only the treated effluent but also all flows received at the plant so as to eliminate overflowing of untreated or partially treated flows at shore.

8.6.1 Discharge Impact Control, Protection and Maintenance

The outfall sewer should be designed to discharge to the receiving stream in a manner acceptable to the reviewing authorities. The designer should contact the local Conservation Authority, the Ministry of Natural Resources (MNR) and review the federal *Navigable Waters Protection Act* for site specific requirements. Consideration should be given in each case to the following:

- Preference for free fall or submerged discharge at the selected site;
- Utilization of cascade aeration of effluent discharge to increase dissolved oxygen; and
- Limited across-stream dispersion as needed to protect aquatic life movement and growth in the immediate reaches of the receiving stream.

The outfall sewer should be so constructed and protected against the effects of floodwater, tide, ice or other hazards as to reasonably ensure its structural stability and freedom from stoppage. A manhole should be provided at the shore end of all gravity sewers that extend into receiving waters. Hazards to navigation should be considered in designing outfall sewers.

8.6.2 Sampling Provisions

All outfalls should be designed so that a sample of the effluent can be obtained at a point after the final treatment process and before discharge to the receiving waters.

8.7 ESSENTIAL FACILITIES

8.7.1 Emergency Power Supply Facilities

The need for standby power and the extent of equipment requiring operation by standby power should be individually assessed for each sewage treatment plant and pumping station. Some of the factors that will require consideration in making the decisions regarding standby power and the processes to be operated by the standby power facility are as follows:

- Reliability of primary power source;
- Number of feeder lines supplying the grid system, number of alternate routes within the grid system and number of alternative transformers through which the power could be directed to the sewage treatment plant;
- Whether sewage enters the plant by gravity or is pumped;
- Type of treatment provided;
- Pieces of equipment which may become damaged or overloaded following prolonged power failure;
- Assimilation capacity of the receiving waters and ability to withstand higher pollution loadings over short time periods; and
- Other uses of receiving water body.

Standby generating capacity may not be needed for aeration equipment used in the activated sludge process in cases where there is no history of long-term (4 hours or more) power outages. However, full power generating capacity is needed for sewage discharges to critical areas of surface water receiver such as upstream or near bathing beaches and water supply intakes.

Continuous disinfection, where required, should be provided during all power outages. Continuous dechlorination is required for those systems that dechlorinate.

Where standby power is not needed for pumping or treatment, the designer should include the provision of a small [typically 25 kW (33.5 hp)] generator set having sufficient capacity to provide the power for lighting and instrumentation, so that in the event of transient power outages, the plant will have sufficient power available for safe operation and to maintain instrumentation.

The standby power equipment should be located so that it connects conveniently into the electrical distribution system of the plant. It should also be close to other potentially noisy equipment, so that adequate acoustic measures need only be taken over small areas. Sufficient fuel storage should be provided, taking into account the historical data on length of power outages in the area and any weather or other conditions that may preclude fresh deliveries of fuel. Where a diesel generator is used, a minimum of:

- 450 L (120 US gal) fuel tank should be provided for generator set capacities of up to 25 to 30 kW (34 to 40 hp);
- 900 L (240 US gal) fuel tank for set capacities from 30 to 100 kW (40 to 134 hp);
- 1135 L (300 US gal) fuel tank for set capacities from 110 to 160 kW (147 to 214 hp); and
- 2 x 1135 L (2 x 300 US gal) fuel tanks for set capacities from 160 to 300 kW (214 hp to 402 hp).

Equipment suppliers should be contacted for actual fuel consumption of their generator units. Fuel storage for both portable and permanent engine generators should be adequate to operate the pump station for a minimum of 12 and preferably 24 continuous hours without refueling. Alternative fuelled standby power equipment (e.g. natural gas) could also be considered.

Either underground or inside fuel storage tanks may be used. In considering which type to use, factors such as corrosion potential, consequences of leakage, required storage volume and the need for fuel pumps should be evaluated.

The designer should refer to *Section 7.7 - Standby Power and Emergency Operation* for additional details.

The location of the standby power system should generally be such that site perimeter noise levels will be in compliance with the ministry *Model Municipal Noise Control By law* and also located so that contaminant levels at the nearest *point of impingement* due to stack emissions are in compliance with the requirements of Section 9 of the *Environmental Protection Act*. (*Section 3.11 - Emissions of Contaminants to Air)*.

8.7.2 Water Supply

An adequate supply of pressurized water should be provided to ensure general cleanliness around the plant. Chemical quality of the water supply should be checked for suitability for its intended uses such as in heat exchangers and chlorinators.

Where a sewage treatment works obtains water from a municipal potable water supply, the supply needs to be protected with a CSA rated, reduced pressure principle backflow preventer[2] at each point of connection with the municipal system. Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:

- Lavatory;
- Water closet;
- Laboratory sink (with vacuum breaker);
- Shower;
- Drinking fountain;
- Eye wash fountain; and
- Safety shower.

Hot water for any of the above units should not be taken directly from a boiler used for supplying hot water to a sludge heat exchanger or digester heating unit.

Where a potable water supply is to be used for any purpose in a plant other than those listed above, a break tank, pressure pump and pressure tank should be provided. Water should be discharged to the tank through an air gap at least 150 mm (6 in) above the maximum flood line or the spill line of the tank, whichever is higher.

[2] Consists of two spring loaded check valves operating in series and a diaphragm-activated, pressure differential relief valve, located between the check valves. Two shutoff valves with test cocks complete the device. Recommended for high health hazard risk where it would be impractical to have an air gap separation. Malfunctioning of this device is indicated by discharge of water from the relief port. The backflow preventers require periodic inspection, maintenance and induce high pressure loss. They cannot be installed below ground level and should be protected from freezing. Space for maintenance and testing should be provided.

Wherever possible, to conserve energy and minimize operating costs, effluent water should be used for water uses not requiring potable water. Such uses as chlorinator-injector water, lawn sprinkling, foam control, flushing water, screening, thickening and dewatering process units wash and incinerator off-gas scrubbing can be supplied with STP final effluent. Fixtures supplied with non-potable water should be clearly marked as such.

Where effluent water is used, a sign should be permanently posted at every hose bib, faucet, hydrant or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

8.7.3 Plant Piping

All piping to be used in sewage treatment plants should be manufactured in accordance with the most recent version of the standards from *Canadian Standards Association* (CSA), *Canadian General Standards Board* (CGSB), *American Society for Testing and Materials Standards* (ASTM), or other internationally recognized organizations. Piping for digester gas, propane, fuel oil and natural gas should further comply with the requirements of *Canadian Gas Association* (CGA) as discussed in *Chapter 16 - Sludge Stabilization*.

In the design of the piping, due allowance should be made for future capacities and also for ease of extending this piping without major disruption of the plant operation. In the general piping arrangement, sufficient space should be provided for piping to be removed and should provide for the proper isolation of pipe sections through valves to enable them to be repaired or replaced.

In larger plants, galleries are often used for the location of process piping and for the passage of operating staff between buildings and tank units. Tunnels may be formed by using the walls of adjacent tank structures and the floor slab may be common to all structures. Although galleries will generally cost more than buried piping systems, their use may be justified due to the more convenient plant operation and maintenance access.

The designer should allow for the possibility that piping could be installed during construction when temperature conditions could be substantially different from the design condition. For example, piping could be installed in temperatures anywhere between -20 °C to +40 °C (-4 °F to 104 °F) and substantial differences in pipe lengths could occur. For this reason the use of PVC pipe with cast iron mechanical joint fittings is not recommended. Where piping is cast in place, due allowance should be made for differential expansion between the pipe material and structures.

Piping should be arranged so that all valves, flow meters and other items which may require regular inspection or maintenance are conveniently accessible. Piping should be provided with drains at all low points and air release valves at all high points. Sludge and scum piping should be provided with cleanouts and facilities to permit water and/or steam cleaning. Scum piping should be smooth walled pipe, preferably glass-lined.

The design of the piping should allow for proper restraint under all anticipated conditions, particularly where surges may occur and high transient pressures could result, or where different temperatures occur seasonally.

Where piping connections are made between adjacent structures, at least one flexible coupling should be provided if there is any possibility that differential settlement could occur. Particular attention should be given to pipe bedding in areas adjacent to structures to avoid settlement damage.

Under the *Industrial Establishments Regulation* (O.Reg. 851) made under the *Occupational Health and Safety Act* (OHSA), piping identification as to flow direction and contents is mandatory only for piping systems containing hazardous substances. However, it is recommended that all piping be adequately identified as to contents and direction of flow so that the operation of the process units is simplified. Piping identification by complete painting of the pipe line and by use of colour banding is recommended. Where there is no existing standard colour coding, it is suggested that the following code be used for pipe colour.

Clearly visible lettering to indicate the actual pipe contents (e.g. raw sludge and waste activated sludge) should be shown on colour bands along with the flow direction arrow. To comply with CSA Standard *CSA B53-58: Code for Identification of Piping Systems*, the bands should be coloured as shown in Table 8-3.

Table 8-3 – Colour Coding for Piping Systems

Classification	**Classification Colour**	**CGSB** (Canadian General Standards Board)	**CIL** (Canadian Industries Limited)
Dangerous Materials	Yellow	505-102	2007
Safe Materials	Green	503-107	94233
Protective Materials	Blue	202-101	95547
Fire Protection	Red	509-102	95557
Gas Piping Controls and Flammable Gas	Orange	508-103	4601-5

The use of paints containing lead or mercury should be avoided. In order to facilitate identification of piping, particularly in the large plants, it is suggested that the different lines be color coded. The following color scheme is recommended:

- Raw sludge line - brown with black bands;
- Sludge recirculation suction line - brown with yellow bands;
- Sludge draw off line - brown with orange bands;

- Sludge recirculation discharge line - brown;
- Digester gas line - orange (or red);
- Natural gas line - orange (or red) with black bands;
- Nonpotable water line - blue with black bands;
- Potable water line - blue;
- Chlorine line - yellow;
- Sulfur dioxide - yellow with red bands;
- Sewage line - gray;
- Compressed air line - green;
- Water lines for heating digesters or buildings - blue with a 150 mm (6 in) red band spaced 760 mm (30 in) apart;
- Fuel oil/diesel - red;
- Plumbing drains and vents - black; and
- Polymer - purple.

The contents and direction of flow should be stenciled on the piping in a contrasting colour.

The designer should ensure compliance with *Code for Digester Gas and Landfill Gas Installations* CAN 1-B105-M81 which states that "all gas piping and controls should be painted or colour coded with high visibility paint and each system of piping should be labeled every linear 3 m (10 ft) with the name of the gas being conducted and the direction of flow".

In sizing, material selection and pressure requirements of piping for use in sewage treatment plants, the following factors should be considered:

- Likelihood of blockage and size of line required;
- Line size required to produce scouring velocities and thus minimize solids deposition and grease buildup;
- Nature of material to be conveyed and suitable piping materials for the application;
- Flow characteristics of material to be conveyed and head requirements of pumps or differential head required for gravity flow;
- Possible settlement and need for support;
- Need for future repair; and
- Need for future removal of pipe sections.

The recommended minimum diameters of piping for various purposes are shown in Table 8-4.

Table 8-4 – Recommended Minimum Pipe Diameters

Type of Flow	Minimum Diameter in mm (inches)
Gravity Flow	
Sewage and Sludge	200 (8)
Pumped Flow	
Sewage	100 (4)
Sludge	150 (6)
Chemicals (non-scale-forming)	12 (0.5)
Chemicals (scale-forming)	25 (1)

8.7.4 Personnel Facilities

The necessity for personnel facilities will be largely dictated by the number of operation and maintenance staff required and the time periods during which the plant is staffed.

As a minimum, it is recommended that provision be made for storage lockers, preferably two for each employee (one for work clothes and one for clean clothes) and a washroom with shower. As the size of the plant and number of staff increases, there will be a requirement to provide more locker space, possibly in a separate change room, a lunchroom which should be of adequate size to serve as a meeting or instruction room for plant staff and a suitable office for plant supervisory staff and record keeping.

Whenever possible, these personnel facilities should be separated from the plant facilities, but with convenient access to the plant. All requirements of OHSA should be included.

8.7.5 Building Services

Adequate heating facilities of a safe type should be provided, with control levels depending on the type of area being heated. In many areas of the plant, sufficient heat need only be provided to prevent freezing of equipment or treatment units.

Buildings should be well ventilated by means of windows, doors, roof ventilators, or other means. All rooms, compartments, pits and other enclosures that are below grade and which need to be entered should have adequate forced ventilation provided when it is necessary to enter them.

Rooms containing equipment or piping should be adequately heated, ventilated and dehumidified, if necessary, to prevent undue condensation. Switches should be provided to control the forced ventilation.

Buildings should be adequately lighted throughout by means of natural light, artificial lighting facilities, or both. Control switches where needed should be conveniently placed at each entrance to each room or area.

As discussed in *Section 9.6 - Security*, it may be advantageous to provide intercom systems between the control centre and other buildings or locations throughout the plant site. In certain circumstances television monitoring may be warranted. Public telephone service should at least be provided to the control centre and other manned centres throughout plant. Empty conduit systems may be provided for future telephone or intercom lines.

Power outlets of suitable voltage should be provided at convenient spacing through plant buildings to provide power for maintenance equipment and extension lighting. Power outlets should also be located at outside locations to permit servicing of such equipment as scraper drive mechanisms, flow meters and comminutors.

Potable water service will be required for most buildings. Reference should be made to *Section 8.7.2 - Water Supply* for requirements relating to backflow prevention for potable water supplies.

8.7.6 Sanitary Facilities

Washrooms with showers and locker facilities should be provided in sufficient numbers and at convenient locations to serve the plant personnel. All requirements of OHSA should be included.

8.7.7 Stairways

Stairways should be installed in lieu of ladders for access to units requiring routine inspection and maintenance, such as digesters, trickling filters, aeration tanks, clarifiers and tertiary filters. Spiral or winding stairs should be used only for secondary access where dual means of exit are provided.

8.8 OPERATOR LICENSING

8.8.1 General

Sewage works are to be operated by persons holding a valid operator's license of the same type as the type for the facility. At least one operator needs to hold a license of the same class or a higher class than the class of the facility and the license needs to be prominently displayed. More detailed information on licensing for operators can be found in the *Licensing of Sewage Works Operators* (O.Reg.129/04) made under the *Ontario Water Resources Act* and the ministry document *"Licensing Guide for Operators of Wastewater Facilities"*.

8.9 SAFETY

8.9.1 General

The following is only a general description of some safety considerations. The designer should refer to all applicable safety codes and regulations, including the *Occupational Health and Safety Act* (OSHA), *Building Code* (O. Reg. 350/06) under the *Building Code Act, 1992, Fire Code* (O. Reg. 388/97) under the *Fire Protection and Prevention Act, 1997* and the *Workplace Safety and Insurance Act* (WSIA). Adequate provision should be made to effectively protect plant personnel and visitors from hazards. The designer should consider the following to satisfy the particular needs of each plant:

- Enclosure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals;
- Hand rails and guards (e.g. kick plates) around tanks, trenches, pits, stairwells and other hazardous structures with the tops of walls less than 1070 mm (42 in) above the surrounding ground level;
- Gratings over appropriate areas of treatment units where access for maintenance is required;
- First aid equipment;
- "No Smoking" signs in hazardous areas;
- Protective clothing and equipment, such as self-contained breathing apparatus, gas detection equipment, goggles, gloves, hard hats and safety harnesses;
- Portable blower and sufficient hosing;
- Portable lighting equipment complying with the requirements of *Electrical Safety Code*, (O.Reg.164/99) made under the *Electricity Act, 1998*;
- Gas detectors listed and labeled for use in Class I, Division 1, Group D locations;
- Appropriately placed warning signs for slippery areas, non-potable water fixtures, low head clearance areas, open service manholes, hazardous chemical storage areas and flammable fuel storage areas;
- Adequate ventilation in pump station areas in accordance with *Section 7.2.10 - Safety Ventilation;*
- Provisions for local lockout on stop motor controls;
- Warning signs should be provided for appropriate areas including excessive noise areas and confined spaces;
- Provisions for confined space entry in accordance with the *Confined Spaces Regulation* (O. Reg. 632/05) under the OHSA; and

- Adequate vector control.

Equipment suppliers and chemical suppliers should also be contacted regarding particular hazards of their products and the appropriate steps taken in the facility design to ensure safe operation.

8.9.2 Hazardous Chemical Handling

The materials utilized for storage, piping, valves, pumping, metering and splash guards should be specially selected with consideration to the physical and chemical characteristics of each hazardous or corrosive chemical that they may come into contact with. Chemical buildings or storage areas should be provided with adequate warning signs, conspicuously displayed where identifiable hazards exist and a storage area for filing *Material Safety Data Sheets* (MSDS) as set out under the federal *Hazardous Products Act* and associated *Controlled Products Regulations*. An MSDS should be available for each chemical. All storage containers should be conspicuously labeled in accordance with the *Workplace Hazardous Materials Information System* (WHMIS) (O. Reg. 860) under OHSA. The WHMIS label includes: the product name, the supplier name, hazard symbol(s), risk, precautionary measures and first aid measures.

8.9.2.1 Secondary Containment

Chemical storage areas should be enclosed in dikes or curbs capable of containing the stored volume until it can be safely transferred. Liquid polymer should be similarly contained to reduce areas with slippery floors, especially to protect travel ways. Non-slip floor surfaces are desirable in polymer-handling areas.

8.9.2.2 Liquefied Gas Chemicals

Properly designed isolated areas should be provided for storage and handling of chlorine, sulfur dioxide and other hazardous gases. Gas detection kits, alarms, controls, safety devices and emergency repair kits should also be provided.

8.9.2.3 Splash Guards

All pumps or feeders for hazardous or corrosive chemicals should have guards, which will effectively prevent spray of chemicals into spaces occupied by personnel. The splash guards are in addition to guards to prevent injury from moving or rotating machinery parts.

All connections (flanged or other types), except those adjacent to storage or feeder areas, should have guards which will direct any leakage away from space occupied by personnel. Pipes containing hazardous or corrosive chemicals should not be located above shoulder level except where continuous drip collection trays and coupling guards will eliminate chemical spray or dripping onto personnel.

8.9.2.4 Piping Labeling

All piping containing or transporting corrosive or hazardous chemicals should be identified with labels every 3 m (10 ft) and with at least two labels in each room, closet, or pipe chase. Color coding may also be used (see *Section 8.7.3 Plant Piping*), but is not an adequate substitute for labeling.

8.9.2.5 Protective Clothing and Equipment

The following items of protective clothing or equipment should be available and utilized for all operations or procedures where their use will minimize injury hazard to personnel:

- Self-contained breathing apparatus recommended for protection against chlorine;
- Chemical worker's goggles or other suitable goggles (safety glasses are insufficient);
- Face masks or shields for use over goggles;
- Dust mask to protect the lungs in dry chemical areas;
- Rubber gloves;
- Rubber aprons with leg straps;
- Hearing protection;
- Rubber boots (leather and wool clothing should be avoided near caustics); and
- Safety harness and line.

8.9.2.6 Warning System and Signs

Facilities should be provided for automatic shutdown of pumps and the sounding of alarms when failure occurs in a pressurized chemical discharge line. Warning signs requiring use of goggles should be located near chemical stations, pumps and other points of potential frequent hazard.

8.9.2.7 Dust Collection

Dust collection equipment should be provided to protect personnel from dusts injurious to the lungs or skin and to prevent polymer dust from settling on walkways which can become slick when wet.

8.9.2.8 Eyewash Fountains and Safety Showers

Eyewash fountains and safety showers supplied with potable water should be provided on each floor level or work location involving hazardous or corrosive chemical storage, mixing (or slaking), pumping, metering, or transportation unloading. These facilities are to be as close as practical to points of chemical exposure. They are to be fully operable during all weather conditions.

The eyewash fountains should be supplied with water of moderate temperature 15 to 32 °C (60 to 90 °F) suitable to provide 15 to 30 minutes of continuous irrigation of the eyes. The emergency showers should be capable of discharging 1.9 to 3.2 L/s (30 to 50 USgpm) of water at moderate temperature and at pressures of 140 kPa to 345 kPa (20 to 50 psi).

8.10 LABORATORY

8.10.1 General

All treatment plants should have a laboratory for making the necessary analytical determinations and operating control tests, except for those plants that utilize only processes not requiring laboratory testing for process control and where satisfactory off-site laboratory provisions are made to meet regulatory monitoring requirements. For plants where a fully equipped laboratory is not required, the requirements for utilities and fume hoods may be reduced. The laboratory should have sufficient size, bench space, equipment and supplies to perform all self-monitoring analytical work and to perform the process control tests necessary for proper management of each treatment process included in the design.

The facilities and supplies necessary to perform analytical work to support industrial waste control programs will normally be included in the same laboratory. The laboratory arrangement should be sufficiently flexible to allow future expansion should more analytical work be performed there in the future. Laboratory size and instrumentation should reflect treatment plant size, staffing requirements and process complexity. Experience and training of plant operators should also be assessed in determining treatment plant laboratory needs.

8.10.2 Categories

Treatment plant laboratory needs may be divided into the following three general categories:

I. Plants performing only basic operational testing; this typically includes pH, temperature and dissolved oxygen;

II. Plants performing more complex operational laboratory tests including biochemical oxygen demand, suspended solids and fecal coliform analysis; and

III. Plants performing more complex operational, industrial pretreatment and multiple plant laboratory testing.

Expected minimum laboratory needs for these three plant classifications are outlined in this section. However, in specific cases laboratory needs may have to be modified or increased due to the industrial monitoring needs or special process control requirements.

8.10.3 Category I: Plants performing only basic operational testing.

8.10.3.1 Location and Space

A floor area of up to 14 m^2 (150 ft^2) should be adequate. It is recommended that this be at the treatment plant site. Another location in the community, utilizing space in an existing structure, owned by the involved authority, may also be acceptable.

8.10.3.2 Design and Materials

The facility should provide for electricity, water, heat, sufficient storage space, a sink and a bench top. The lab components need not be of industrial grade materials. Laboratory equipment and glassware should be of types recommended by American Public Health Association (APHA), American Water Works Association (AWWA) & Water Environment Federation (WEF), *Standard Methods for the Examination of Water and Wastewater, 21st Edition,* as amended.

8.10.4 Category II: Plants performing more complex operational laboratory tests including biochemical oxygen demand, suspended solids and fecal coliform analysis.

8.10.4.1 Location and Space

The laboratory size should be based on providing adequate room for the equipment to be used. In general, the laboratories for this category of plant should provide a minimum of 28 m^2 (300 ft^2) of floor space. Adequate bench space for each analyst should be provided. The laboratory should be located at the treatment plant site on ground level. It should be isolated from vibrating, noisy or high-temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

8.10.4.2 Design and Materials

Floor surfaces should be fire resistant and highly resistant to acids, alkalis, solvents and salts. The cabinets and shelves selected may be of wood or other durable materials. Bench tops should be of acid resistant laboratory grade materials for protection of the underlying cabinets. Glass doors on wall-hung cabinets are recommended.

Fume hoods should be provided for laboratories in which required analytical work results in the production of noxious fumes. Air intake should be balanced against all exhaust ventilation to maintain an overall positive pressure relative to atmospheric in the laboratory. A laboratory grade sink and drain trap should be provided. Laboratories should be air conditioned. In addition, separate exhaust ventilation should be provided.

An analytical balance of the automated digital readout, single pan, 0.1 milligram sensitivity type should be provided. A heavy special-design balance table which will minimize vibration of the balance is recommended.

Laboratories should provide the following: first aid equipment, protective clothing and equipment (e.g. goggles, safety glasses, full face shields and gloves), fire extinguishers, chemical spill kits, posting of "No Smoking" signs in hazardous areas and appropriately placed warning signs for slippery areas, non-potable water fixtures, hazardous chemical storage areas and flammable fuel storage areas.

Eyewash fountains and safety showers supplied with potable water should be provided in the laboratory (*Section 8.9.2.8 - Eyewash Fountains and Safety Showers)*.

8.10.5 Category III: Plants performing more complex operational, industrial pretreatment and multiple plant laboratory testing.

8.10.5.1 Location and Space

The laboratory should be located at the treatment plant site on ground level, with environmental control as an important consideration. It should be isolated from vibrating, noisy, high-temperature machinery or equipment that may have adverse effects on the performance of laboratory staff or instruments.

The laboratory facility needs for Category III plants should be described in the engineering report or facilities plan. The laboratory floor space and facility layout should be based on an evaluation of the complexity, volume and variety of sample analyses expected during the design life of the plant including testing for process control, industrial pretreatment control, user-charge monitoring and effluent quality criteria and monitoring requirements.

Consideration should be given to provide separate (and possibly isolated) areas for some special laboratory equipment, glassware and chemical storage. The analytical and sample storage areas should be isolated from all potential sources of contamination. It is recommended that the organic chemical facilities be isolated from other facilities. Adequate security should be provided for sample storage areas. Provisions for the proper storage and disposal of chemical wastes should be provided.

8.10.5.2 Design and Materials

Floor surfaces should be fire resistant and highly resistant to acids, alkalis, solvents and salts.

Two exit doors should be located to permit a straight exit from the laboratory, preferably at least one to the outside of the building. Panic hardware should be used. Exit doors should have large glass windows for easy visibility of approaching or departing personnel.

Wall-hung cabinets are useful for dust-free storage of instruments and glassware. Units with sliding glass doors are recommended. A reasonable proportion of cupboard style base cabinets and drawer units should be provided.

All cabinet shelving should be acid resistant and adjustable. The laboratory furniture should be supplied with adequate water, gas, air and vacuum service fixtures, traps, strainers, plugs and tailpieces and all electrical service fixtures.

Bench tops should be constructed of materials resistant to damage from normally used laboratory reagents. Generally, bench-top height should be 915 mm (36 in). However, areas to be used exclusively for sit-down type operations should be 760 mm (30 in) high and include kneehole space.

Fume hoods should be located where air disturbance at the face of the hood is minimal. Air disturbance may be created by persons walking past the hood, by heating, ventilating, or air-conditioning systems and by drafts from opening or closing a door.

One sink should be provided inside each fume hood. A cup sink is usually adequate.

All switches, electrical outlets and utility and baffle adjustment handles should be located outside the hood. Light fixtures should be explosion-proof.

Twenty-four hour continuous exhaust capability should be provided. Exhaust fans should be explosion-proof. Exhaust velocities should be checked when fume hoods are installed.

Canopy hoods should be installed over the bench-top areas where hot plate, steam bath, or other heating equipment or heat-releasing instruments are used. The canopy should be constructed of heat and corrosion resistant material.

The laboratory should have a minimum of two sinks (not including cup sinks). At least one of them should be a double-well sink with drain boards. Additional sinks should be provided in separate work areas as needed and identified for the use intended.

Sinks and traps should be made of epoxy resin or plastic materials highly resistant to acids, alkalis, solvents and salts and should be abrasion and heat resistant, non-absorbent and light weight. Traps should be made of glass, plastic, or lead when appropriate, and easily accessible for cleaning. Sewage openings should be located toward the back so that a standing overflow will not interfere.

All water fixtures on which hoses may be used should be provided with reduced zone pressure backflow preventers to prevent contamination of water lines.

Laboratories should be separately air conditioned, with external air supply for one hundred percent make-up volume. In addition, separate exhaust ventilation should be provided. Ventilation outlet locations should be remote from ventilation inlets. Consideration should be given to providing dehumidifiers.

An analytical balance of the automatic, digital readout, single pan, 0.1 milligram sensitivity type should be provided. A heavy special-design balance table which will minimize vibration of the balance is needed

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

Reagent water for analytical requirements using an all-glass distillation system should be supplied to the laboratory. Some analyses require deionization of the distilled water. Consideration should be given to softening and/or deionizing the feed water to the still.

Natural or LP gas (liquefied petroleum gas - propane) should be supplied to the laboratory. Digester gas should not be used. Adequately-sized vacuum lines should be provided, with outlets available throughout the laboratory.

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CHAPTER 9

INSTRUMENTATION AND CONTROL

CHAPTER 9

INSTRUMENTATION AND CONTROL

This chapter provides information on instrumentation that is generally utilized in sewage treatment plants. However, instrumentation is evolving all the time and enhancements are ongoing - therefore information in this chapter should be used as a general guideline for instrumentation selection and use. Controls for sewage treatment plants are also presented in terms of the general function and considerations for use.

9.1 GENERAL

The requirements for instrumentation and control will depend on the size of the *sewage works*, design standards and philosophy regarding instrumentation and control (I&C) and the type of processes employed. In general, instrumentation and control should allow for safe and efficient manual and automatic operation of all parts of the plant, with minimum operator effort. All automatic controls should be provided with manual back up systems.

Where some parts of the plant may be operated or controlled from a remote location, local control stations should be provided and should include the provision for preventing operation of the equipment from the remote location. Consideration should be given to providing communication via intercom between remote stations and the local stations. In some cases, the use of television equipment may be justified to provide scanning functions of local instrumentation control centers as well as process equipment.

Decisions will have to be made by the designer in conjunction with the owner and operations staff as to which equipment will be controlled locally and which will be controlled from a remote location and whether control will be automatic or manual. For instance, at a small sewage treatment plant, scum pumping may be controlled locally and manually, whereas raw sewage pumping should be automatically controlled, regardless of plant size. In addition, the points of control and the type of primary device should be selected. Decisions need to be made as to whether the instrumentation is to totalize, indicate and/or record and whether alarm functions are to be incorporated.

In making decisions relating to instrumentation and control, the following factors should be considered:

- Plant size;
- Effluent requirements;
- Plant process complexity;
- Hours in the day that the plant will be staffed;
- Potential chemical and energy savings with automation;

- Reliability of primary devices for parameter measurement;
- Preferred location for primary device;
- Parameters with useful significance to process;
- Equipment which should be controlled automatically;
- Equipment which should be controlled manually;
- Equipment which should be controlled remotely;
- Equipment which should be locally controlled;
- Owner's design standards and philosophy for I&C and automation;
- Data requiring display at the control centre; and
- Indication, totalizing and recording functions necessary to the overall process.

For proper operation of sewage treatment plants, the following parameters should be measured:

- Sewage flow rates, including raw sewage, bypassed flows and flows through plant subsections (flow trains);
- Chlorine dosage;
- Sludge pumping rate, including raw, digested sludges and activated sludge return;
- Digester supernatant flows;
- Sludge dewatering return stream flows, where applicable;
- Chemical dosage for phosphorus removal processes;
- Anaerobic digester gas production and utilization;
- Anaerobic digester temperature;
- Hazardous gas levels; and
- Chlorine residual (if chlorine used for disinfection).

Auxiliary instrumentation may be considered to measure the following parameters:

- Air flows;
- Mixed liquor dissolved oxygen concentrations;
- Mixed liquor and return activated sludge suspended solids concentrations;
- Sludge blanket levels;
- Sludge concentrations; and
- UV intensity.

9.2 TYPE OF INSTRUMENTS

The type of instruments that will be required to measure the parameters mentioned above are classified as prime or primary element devices which transform a signal from the physical process to a suitable signal for transmission via a transmitter. These devices are broken down into various measurement parameters and each of the parameters is further broken down into features and considerations with a brief description of particular process applications.

Proper design of the instrumentation installation needs to be taken to ensure the instrument accuracy, including avoiding interference from other equipment or aspects of the plant. Flow instrumentation generally requires upstream and downstream straight lengths to avoid non-uniform flow in the vicinity of the instrument. Pipe flow instruments need to be located to avoid interference from valves and bends. Considerations for all instrumentation need to ensure the instrument accuracy by following good design practices. The instrument manufacturer's specifications, requirements and expertise should be followed.

9.2.1 Flow Elements

Flow meter accuracy needs to account for the accuracy of the primary and secondary devices. Generally, the primary device is the physical structure in which the measurement is being made; an example is a flume in an open channel. The secondary device is generally the measuring or electronic device, such as the level sensor, for example an ultrasonic level device, on a flume. The combined accuracy of the entire flow measuring structure is often referred to as the system accuracy. A main flow measuring device such as the influent or effluent flow meter at a sewage treatment plant (STP) should have a minimum 10% system accuracy.

For the main flow device at an STP, consideration should be given to a secondary means to confirm the accuracy of the instrument, as these flow instruments are critical to determining plant *capacity*, upgrade or expansion requirements and for proper operations. These can include redundant flow instruments and/or the ability to conduct draw-and-fill tests or secondary checks of the physical primary flow devices. Physical primary flow devices that can be measured independently of the secondary device include flumes (e.g. Parshall flumes), weirs (e.g. V-notch or sharp crested weirs) or Venturi tubes. For these primary devices, a separate physical measurement can be made to confirm the accuracy of the flow instrument. Physical checks of this type need to take into account the proper installation of the primary structure.

Flow measurements should be conducted throughout the STP. Some areas are critical for gauging plant performance and capacity, others are key for plant operations, and some are useful for plant optimization and minimizing operating costs (e.g. electrical and chemical usage). Critical flow measurement areas include STP influent, effluent, overflows and bypasses.

Key flow measurement areas related to plant operations include:

- Unit process or plant influent and effluent streams;
- Biological return streams, e.g. return activated sludge;
- Raw and biological sludge wasting; and
- Recycle streams such as centrate and filtrates.

Useful plant optimization flow measurements include:

- Chemical and polymer usage;
- Septage and/or leachate additions, where applicable;
- Mechanical thickening and dewatering flows; and
- Gas flows, including aeration and biogas.

9.2.1.1 Magnetic Flow Meter

The magnetic flow meter operates on the principle that a conductor passing through a magnetic field will produce a DC voltage directly proportional to the speed of the conductor. The following points generally apply to magnetic flow meters:

- Preferably installed with slight incline with upward flow;
- The meter needs to be full for proper operation;
- Should have a minimum velocity of approximately 2 to 4 m/s (6.6 to 13 ft/s);
- Piping arrangement error is minimal and is not a substantial factor;
- Avoid use where entrained gases are present; and
- Accuracy approximately 1% of rate.

9.2.1.2 Parshall Flume

A Parshall flume operates on the principle of a known relationship between the upstream liquid depth and flow, under a range of operating conditions. A Parshall flume is the most common flume used in sewage treatment plants. The following points generally apply to Parshall flumes.

- Flume should be level;
- Accuracy is affected by upstream channel arrangement (should have at least ten channel widths); and
- Accurate to within ±5 % of the measured rate.

9.2.1.3 Ultrasonic Flow Meter

Ultrasonic flow meters are used in full-flowing pipes, often to measure sludge flows. The following points generally apply to ultrasonic flow meters:

- No contact with medium being measured;
- If doppler type, then the presence of entrained gases or solids are required for proper operation;
- If grease is present in the medium, then heat or ultrasonic cleaning should be used;
- Operating velocity 1 to 10 m/s (3.3 to 33 ft/s); and
- Accurate to within ±1 to 2% of the measured rate.

9.2.1.4 Venturi Tube

The Venturi tube operates on the principle that the pressure differential between the inlet and the throat is proportional to the square of the flow. The following points generally apply to Venturi tubes:

- Meter should be full for proper operation;
- Accuracy is affected by upstream piping arrangement;
- If used on sludge of any kind, then water purge arrangement should be used;
- Accuracy to within ±2% of the measured rate; however, accuracy will be much lower at low flows, depending on which type of transmitter is used;
- If time differential type, then the presence of entrained gases and/or solids will affect the accuracy severely;
- Accuracy affected by upstream and downstream piping arrangement; and
- Accurate to within ±2% of the measured rate.

9.2.1.5 Volumetric Meter

The volumetric flow meter can measure flows in pumping stations controlled by low and high level contacts.

It does not require cleaning or recalibration as no parts are in contact with sewage and can be installed remotely from the station. The measuring principle is flow rate equals volume divided by time.

- Programmed with on-site wet well volumes;
- Digital display of inflow and outflow rates, total flow and pumping time; and
- Accurate to within ±0.1% of the measured rate.

Caution should be used where variable speed pumping is employed.

9.2.1.6 Rotameter

The Rotameter is a tapered tube that has a ball float permitting rough visual readings. Mostly used in chlorinators and/or ammoniators where it is a standard fixture. Since it is installed in the flow line, it needs to be line sized and has limitations when transmission of a signal is required.

9.2.1.7 Gas Meters

Most common gas flows at a sewage treatment plant are for aeration tank air flows and digester gas flows. The following instruments are commonly used:

- Mass flow;
- Orifice plate;
- Rotary positive displacement; and
- Turbine.

9.2.2 Level Elements

The following points generally apply to the level elements presented.

9.2.2.1 Ultrasonic

- No contact with medium being measured;
- Accuracy to within ±2% of actual reading;
- Typically the sensor should be mounted a minimum distance above the high liquid level and should be located away from tank walls or other obstructions that may cause false echoes; and
- Not recommended in locations where foam is dense and persistent.

9.2.2.2 Float Type

- Range of 0 to 11 m (0 to 36 ft);
- Accuracy to within ±1% of actual reading;
- Normally located in a stilling well where turbulence is expected; and
- Commonly used for high and low level alarms and for controlling pump starts and stops.

9.2.2.3 Capacitance Probe

- May be used in applications that require continuous level measurement and also as switches for alarms or start/stop control.

9.2.2.4 Bubbler

- Not common in new installations;
- Range 0 to 56 m (0 to 184 ft); and
- Accuracy to within ±0.1% of actual reading; however, the accuracy will be further affected by the type of transmitter selected.

9.2.3 Other Analytical Instruments

9.2.3.1 Density Elements

The following points generally apply to density elements:

Radiation Type

- Accuracy 0 to 15%;
- Can be difficult to keep in calibration and requires frequent maintenance; and
- Requires certified personnel to handle radioactive material.

Ultrasonic

- Range 1 to 10%;
- Accuracy repeatability is ±0.5%; and
- Avoid use where entrained gases are present.

9.2.3.2 Dissolved Oxygen Measurement (Galvanic)

- Installation details are generally related to the choice of placement of the analyzer in the process fluid; and
- Analyzers generally require frequent maintenance.

9.2.3.3 Suspended Solids Measurement

- Installation details for these instruments are unique to each manufacturer;
- Two main types are turbidity and optical;
- Turbidity analyzers are recommended for applications involving suspended solids; and
- Optical analyzers are recommended for applications involving solids concentrations from 20 mg/L to 8% solids.

9.2.3.4 Pressure Elements

The following points generally apply to pressure elements:

Bellow (Lower Pressures)

- Pressure range 0 to 2,000 kPa (0 to 290 psi);
- Accuracy to within ±1% of actual reading; and
- Installation should include the use of block and bleed valves.

Bourdon Tube (Higher Pressures)

- Pressure range 0 to 35,000 kPa (0 to 5080 psi);
- Accuracy ±1% of full scale; and
- Installation should include the use of block and bleed valves.

Liquid- to-Air Diaphragm (Commonly used in sensing pressures involving corrosive chemicals)

- Pressure to approximately 20 m water column or 0 to 3500 kPa (0 to 510 psi);
- Accuracy ±1% of scale;
- Installation should include the use of block and bleed valves; and
- Temperature extremes should be avoided and location should be as close as possible to the process measurement site.

Liquid-to-Liquid Diaphragm

- Pressures to approximately 20 m (65 ft) water column or 0 to 3500 kPa (0 to 510 psi);
- Accuracy ±1% of scale;
- Installation should include the use of block and bleed valves; and
- Temperature extremes should be avoided and location should be as close as possible to the process measurement site.

Strain Gauge (Commonly used in conjunction with a bellow)

- Should have temperature compensation;
- Accuracy ±1% of reading;
- Not sensitive enough for low pressure ranges.

9.2.3.5 Sludge Blanket Detector

There are basically two types of sludge blanket level detectors available; one is the photocell type and the other is the ultrasonic type. If the application requires an ON/OFF type of control, then the photocell type may be suitable. However, if an analog type of control or monitoring is required, then the ultrasonic type will be required.

9.2.3.6 Temperature Elements

All temperature elements should be selected with care to assure that the appropriate device is chosen for a given temperature range. The following points generally apply to temperature elements:

Gas Filled System

- Most common in sewage treatment plant application;
- Temperature range 0 to 100 °C (32 to 212 °F); and
- Accuracy ±1% full scale.

Resistance Temperature Detector

- Use of thermowell advised;
- Temperature range 0 to 300 °C (32 to 572 °F); and
- Accuracy to within ±0.5% of actual reading.

Thermocouple

- Use of thermowell advised;
- Temperature range approximately 0 to 1000 °C (32 to 1832 °F); and
- Accuracy ±1% full scale.

Thermistor

- Use of thermowell advised; and
- Temperature range approximately 0 to 300 °C (32 to 572 °F).

9.3 PROCESS CONTROLS AND INSTRUMENTATION

9.3.1 Pumping Stations

Pumping stations require dependable and simple instrumentation and controls. The parameters that require monitoring and control in a pumping station are: level, flow, pumps, motors and alarms.

9.3.1.1 Level Control

The purpose of a level control is to regulate the pumping rate of sewage to the treatment plant. If the pumps are variable speed an analog monitoring and control system should be used. If the pumps are constant or multiple speeds, a stepping type of control system should be used.

9.3.1.2 Flow Monitoring

The flow metering device should be selected very carefully to ensure that there are no obstructions where clogging could potentially occur. Routine preventative maintenance should also be considered in the selection of the flow meter.

9.3.1.3 Pumps and Motors

The following parameters should be monitored:

Pump

- Bearing temperature;
- Casing temperature;
- Vibration;
- Speed; and
- Suction and discharge pressures.

Motors

- Voltage;
- Current;
- Hours of operation;
- Bearing temperature; and
- Windings temperature.

9.3.1.4 Alarms

Alarms are the final warning to the operator that the system is malfunctioning and unless corrective action is taken, damage (e.g. flooding, bypassing and/or equipment damage) may occur. Audio/visual alarms are recommended in order to focus the operator's attention on the actual fault condition.

A pumping station should have at least the high and low liquid level alarm and other pump alarms, including motor winding temperature, pump and motor bearing temperature and motor overload.

For further protection of motors in heavy-duty applications, a shock-loading relay can be installed to protect against unexpected motor overloads.

9.3.2 Mechanical Bar Screens

There are two methods to control mechanical bar screens:

- Simple manual start or stop switch requires the presence of an operator at the screen; and
- Automatic start or stop switch operates when activated by a differential pressure switch. The screen should run for at least one complete screen revolution. To achieve the minimum running time, a timer or limit switch may be used. In addition a timer should be provided to ensure

periodic cleaning of the screen, regardless of actual headloss. A means to initiate a cycle manually should be provided to permit the operator to cycle the unit as required if the automatic systems fail.

9.3.2.1 Alarms

There should be a differential pressure switch for alarm signal with a head loss setting of approximately 100 mm (4 in) higher than the setting for automatic start up of the mechanical bar screen.

9.3.3 Primary Treatment

9.3.3.1 Raw Sludge Pumping

The raw sludge pumps should be set up in such a way that the following features are incorporated:

- Selectable pump duty (manual selection of duty pump);
- Manual override of automatic controls;
- Individually selected hopper pumping times;
- Adjustable density control;
- Sludge flow; and
- Sludge density.

9.3.3.2 Scum Pumping

The scum pumps should be set up in such a way that the following features are incorporated:

- Selectable pump duty;
- Manual override of automatic controls;
- Automatic control system consisting of high and low scum level switches;
- Scum temperature indicator; and
- The duty pump should start at scum high level and stop at scum low level in the scum tank.

9.3.4 Secondary Treatment

9.3.4.1 Aeration Tank Dissolved Oxygen

Automatic dissolved oxygen (DO) control systems can be used to control the rate of air supply to the aeration tanks. The use of DO control can result in energy and operating cost savings. There are several different methods of automatic DO control. The most commonly used are:

- **Flow Ratio** - This consists of a fixed ratio of air volume to plant influent flow; it should be noted that this control strategy does not account for load variations that are not associated with flow changes.
- **Closed Loop Control** - This consists of DO probes and controllers where the actual DO reading is compared with a set point on the controller and the resultant deviation signal is used to increase or decrease the oxygen supply to the aeration tanks.

There are other methods, such as food-to-microorganism (F/M) ratio or solids retention time control. These methods require the use of a computer for calculation, forecasting and modelling.

9.3.4.2 Chemical Control Systems

Chemical addition with the exception of the chlorination system is a feed forward control system. This consists of a feeder or chemical metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control.

Chlorine addition is a compound-loop control system that consists of adjustable ratio of chlorine to influent or effluent flow with trim based on chlorine residual as measured by a chlorine analyzer.

The chlorine addition for larger treatment plants should consist of at least three chlorinators and two analyzers:

- One chlorinator for pre-chlorination;
- One chlorinator for post-chlorination;
- One chlorinator standby that can be used as either pre- or post-chlorination;
- One analyzer for post-chlorination; and
- One analyzer for pre-chlorination.

Each analyzer should be capable of being switched to the standby chlorinator.

Dechlorination chemical addition will have similar requirement for control. Dechlorination control is generally based on chlorine residual after dechlorination, with a set point and alarm level identified.

9.3.5 Tertiary Treatment

Control and monitoring for tertiary treatment processes will depend on the process being used. This could include, for example, tertiary clarifiers, filters or membranes. Monitoring and control instrumentation could include:

- Water level;

- Influent and effluent turbidity or solids; and
- Pressure.

9.4 PROCESS NARRATIVE AND BASIS OF CONTROL

The designer should prepare a report which provides a process narrative for the STP and that briefly describes each component of the plant, including the raw influent quality, pretreatment processes, biological treatment processes, pumping equipment, solids handling processes, instrumentation, monitoring and sampling equipment, as applicable. The report should also identify and explain the basis of control for the system.

Process and instrumentation diagrams (P&ID) should be developed for all sewage treatment facilities and should include all major and minor processes along with all ancillary process equipment.

Control systems should be designed with a user-friendly, human-machine interface (HMI) system to facilitate plant operation and on-line monitoring. Equipment status, flow rates, water levels, pressures and chemical feed rates should all be displayed via an HMI. All automated systems should be designed with a manual override some other form of redundancy to allow safe operation in the event of a hardware or communication failure.

9.5 CONTROL AND MONITORING SYSTEMS

Two control and monitoring systems are available. One is the conventional system with recorders, indicators, switches, push buttons, indicating lights and control panels and the other is the computerized system.

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions. The computerized system is a multi-purpose system with limited scope for modification or a dedicated-purpose system with standard hardware and customized software.

Both computerized systems have two basic configurations:

- A centralized configuration where all intelligence is resident in the computers in the Central Control Station; and
- A distributed control configuration where the intelligence is distributed throughout the system.

The distributed system hardware costs will be higher than the centralized configuration; however, wiring and installation costs will be less. There are several important advantages to the distributed system.One of the advantages is that with intelligence distributed throughout the system, the software required for the computers at Central Control becomes less complicated and requires less maintenance. The intelligence contained in the other components of the system will be on firmware which requires no maintenance. Another advantage is that in the event of a communication failure, each intelligent component in the system can operate on its own and maintain some pre-programmed condition based on its own sensors. Therefore, when a lower intelligence component loses communication with a higher intelligence component, it will still function with the pre-determined fail safe program to maintain system operation.

The computerized systems can be arranged so that all operating decisions can be made by the computer based on instructions given at an earlier stage of the formal programming. Alternatively, the terminal equipment can be used for information display and manual initiation of control commands, that is as a remote manual control station.

A programmable logic controller (PLC) based system is a multi-purpose system with extensive scope for modification. The plant status, alarms, motor starters, meters and analyzers are all wired into input/output (I/O) cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The I/O racks are associated with controllers that are programmed to perform the required process control functions. Adjustments can generally be made easily by modification of or addition to the PLC programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of an HMI. The HMI may be dedicated hardware and software or may come in the form of personal computers utilizing HMI software and connected to the PLC communications system. These systems vary widely in their capabilities and performance. The selection of hardware and software should be done carefully to assure proper current performance and future supportability and expendability.

9.5.1 Maintainability

Sewage treatment plants are becoming more dependent on control systems of all types and complexities. STPs are becoming increasingly dependent on the one common feature of control systems, this being the software. Without proper documentation and maintenance of the software, proper operation of the plant is at risk. Plant operation relies on proper application programs, which could be lost without adequate system documentation.

System backup programs may also be at risk if system activities such as changes to program logic, the tuning parameters and instrument installation are not properly documented. Maintenance of the control system is difficult if not impossible to accomplish without proper documentation.

9.5.2 Identifying the Required Documents

The operation and maintenance of an STP that uses any type of programmable device for process control requires the following types of documents:

- System description in narrative format;

- System block-diagram drawing that identifies location and node names of the connected PLCs, PCs, operator interfaces, servers and modems;
- Software used for system configuration is always updated and ready to load;
- Drawings showing I/O wiring connections and address assignments;
- Address assignments identifying all of the variables within the control system, such as register and address assignments, variables and I/O tables (if required);
- Control system programs for each PLC or programmable process control device in a state that is updated and ready to load, as well as a printout of the program; and
- Narrative description of each part of the program and the software used to enter the description.

9.5.3 Smart Instrumentation

Instruments that provide the control system with both process measurements and diagnostic information about the instrument are referred to as "smart instruments." Both pieces of information are critical in today's control systems due to the way data is moved and used. It is common to move analytical data from the control system to a server where many people can view the data and use it in reports. If the instrument is malfunctioning, the data may be in error, but it will be used in reports generated from the server. Smart instruments can provide an indication of the quality of the data in question and, therefore, whether reports are accurate.

9.5.4 PLC Documentation Software

Specifications for STPs using PLCs should include comprehensive requirements for PLC documentation software. Documentation systems, either from the PLC manufacturer or third-party software vendors, should provide functions important to maintaining a plant such as uploading, verifying and storing the application programs.

9.5.5 Reliability and Maintenance Considerations

An instrumentation and control system should be designed with both operational reliability and maintainability if it is to properly serve its purpose. To assist in review of this vital requirement, the following list of frequent design oversights, errors and omissions has been compiled. The list does not contain any solution to these problems. It is intended only as a reminder to designers or checkers. Solutions depend on conditions or factors unique to specific projects.

- Millivolt-level signals inadequately separated or shielded from parallel runs of heavy power circuits;

- Millivolt-level signals not in twisted shielded pair or triad construction;
- Electric and pneumatic signal conductors not in conduit or otherwise protected from physical/mechanical damage;
- 120 VAC control circuits too long, allowing distributed capacitance to keep the circuit energized after the primary control element is opened;
- Hazardous area (refer to U.S. *National Electric Code* (NEC) section 500);
- Failure to use oil-free air in pneumatic control systems;
- Failure to indicate when single-point grounding is required;
- Failure to indicate or specify required voltage regulation or over-voltage protection;
- Failure to specify adequate equipment enclosures for adverse, hostile or hazardous environments;
- Failure to consider possible or probable clogging of sensor lines by grease or solids in the process stream;
- Failure to specify or provide isolation valves on instruments connected to process piping;
- Failure to specify snubbers on pressure switches;
- Failure to provide needle valves for control of operating air or hydraulics to control valves;
- Float switches in highly turbulent areas;
- Flow meters too close to bends in process pipes;
- Installation of equipment in areas difficult or impossible to reach for maintenance;
- Failure to consider operator convenience in layout or design of control system; and
- Failure to provide operator with sufficient process data.

9.6 SECURITY

Site security has historically been to protect property, protect staff and prevent endangering the public. Recent events have shown that in addition to these issues, sewage treatment works are also potential targets of malevolent acts of destruction and disruption from domestic and international terrorists. Purposeful contamination of sewage as well as damage to or destruction of treatment or conveyance systems, can lead to widespread and long-term environmental damage and serious public health impacts. For additional details on security issues refer to ASCE, AWWA and WEF (2004), *Interim Voluntary Security Guidance for Wastewater/Stormwater Utilities*.

The security systems covered in this section refer specifically to electronic type surveillance and intruder alert systems. Fencing and other safety aspects are covered elsewhere.

In larger sewage treatment plants, the main gate should have at least one of the following access control systems:

- Punched or magnetic cards with a card reader at a central control station;
- Closed circuit TV (CCTV) system where the operator has to operate the gate/door from a remote location; and
- An intercom system where the operator has to operate the gate/door from a remote location.

Legal/illegal entry alarm systems should be provided in remote pumping stations and if required in plant buildings. These systems should include door and window switches and tapes that will provide indicator signals to a central location that an entry has been made.

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CHAPTER 10

PRELIMINARY TREATMENT

CHAPTER 10

PRELIMINARY TREATMENT

This chapter describes those processes, generally located at the headworks of a sewage treatment plant, that are designed to remove debris from sewage, to protect equipment and downstream processes. The preliminary treatment processes described in this chapter are screening, comminution, grinding and grit removal. A section is also included on preaeration and flow equalization.

10.1 SCREENING DEVICES

Screens should be placed in the influent flow at the headworks of the sewage treatment plant (STP) to remove debris that may harm other process units.

Coarse screens, or trash racks, should be provided as the first treatment stage for the protection of plant equipment against blockage or physical damage. Trash racks can precede finer screens when serving combined sewer systems. Coarse screening can be provided in the form of bar screens (manually or mechanically cleaned). Table 10-1 provides the approximate screen opening sizes for various classifications of screens.

Table 10-1 – Screen Size Openings for Various Screen Classifications

Screen Classification	Screen Opening Size (Range)
Trash Rack	> 25 mm (>1 in)
Coarse	6 – 25 mm (1/4 - 1 in)
Fine	1 – 6 mm (1/25 – 1/4 in)
Microscreen	< 1 mm (<1/25 in)

When considering which types of screening devices should be used (whether manually or mechanically cleaned), the following factors should be considered:

- Effect on downstream treatment and sludge disposal operations;
- Possible damage to comminutor or barminutor devices caused by stones or coarse grit particles;
- Head losses of the various alternative screening devices;
- Impact of debris loading on screen (including leaf loading in the case of combined sewer systems);
- Maintenance and labour requirements; and

- Screenings disposal requirements.

Facilities for the removal, drainage, washing, storage and ultimate disposal of accumulated screenings should be provided when manually or mechanically cleaned screens are used.

10.1.1 Coarse Screens

It is recommended that protection for pumps and other equipment be provided by upstream trash racks, coarse bar racks, or coarse screens.

10.1.1.1 Design and Installation

In general, manually cleaned screens should be placed on a slope of 30 to 45º from the horizontal. Approach velocities should be between 0.4 m/s (1.25 ft/s) to prevent settling and 0.9 m/s (3.0 ft/s) at design average daily flow, to prevent forcing material through the openings.

Dual channels should be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions should also be made to facilitate dewatering each unit or channel for maintenance or repair. The channel preceding and following the screen should be shaped to eliminate stranding and settling of solids. The screens serving combined sewer system should be provided with a bypass or equipment to remove the screen to prevent flooding in the event of screen blinding due to excess material like leaves.

Where a single mechanically cleaned screen is used, an auxiliary manually cleaned screen should be provided. Where two or more screens are present, the *capacity* should be provided to treat design peak hourly flow with one unit out of service.

The screen channel invert should be 75 to 150 mm (3 to 6 in) below the invert of the incoming sewer. Entrance channels should be designed to provide equal and uniform distribution of flow to the screens.

The effect of changes in backwater elevation, due to intermittent cleaning of screens, should be considered in the location of flow measurement equipment (*Section 9.2.1 - Flow Elements*).

Screening devices and screenings storage areas should be protected from freezing. A convenient and adequate means for removing screenings should be provided. Hoisting or lifting equipment should be provided. Provision should also be made for washing of screenings.

Facilities are recommended to be provided for handling, storage and disposal of screenings in an acceptable manner in accordance with applicable requirements. Final disposal to landfill generally requires that the screenings meet a limit in terms of dry solids, slump, or presence of a free liquid. (*Section 18.5.1 - Municipal Solids Waste Landfilling*). Separate grinding of screenings and return to the sewage flow is not recommended.

Manually cleaned screening facilities should include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities are recommended to be provided for both the platform and the storage area.

10.1.1.2 Access and Ventilation

Screens located in pits more than 1.2 m (4 ft) deep need to be provided with stairway access. Access ladders are acceptable for pits less than 1.2 m (4 ft) deep, in lieu of stairways.

Screening devices, installed in a building where other equipment or offices are located, need to be isolated from the rest of the building. These devices should be provided with separate outside entrances and be provided with separate and independent fresh air supply.

Fresh air needs to be forced into enclosed screening device areas or into open pits more than 1.2 m (4 ft) deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required, at least 12 complete air changes per hour are recommended. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour are recommended when personnel enter the area. The air change requirements are based on 100 percent fresh air.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment is recommended to be interconnected with the respective pit lighting system. It is recommended that the fan wheel be fabricated from non-sparking material. Explosion proof gas detectors need to be provided.

10.1.1.3 Safety and Shields

Manually cleaned screen channels need to be protected by guard railings and deck gratings, with adequate provisions for removal or openings to facilitate raking.

Mechanically cleaned screen channels need to be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair. Mechanical screening equipment is recommended to have adequate removable enclosures to protect personnel against accidental contact with moving parts and to prevent dripping in multi-level installations.

A positive means of locking out each mechanical device and temporary access for use during maintenance is recommended. Floor design and drainage should be provided to prevent slippery areas. Suitable lighting should also be provided in all work and access areas.

10.1.1.4 Electrical Equipment and Control Systems

It is recommended that all mechanical units which are operated by timing devices be provided with auxiliary controls which will set the cleaning mechanism in operation at a preset high water elevation. If the cleaning mechanism fails to lower the high water, an alarm should be signaled.

Electrical equipment, fixtures and controls in the screening area where hazardous gases may accumulate needs to meet the requirements of the *Electrical Safety Code for Class I, Zone 1*, Group D locations (O.Reg. 164/99 made under the *Electricity Act, 1998*).

It is recommended that automatic controls be supplemented by a manual override at the location of the equipment.

10.1.2 Fine Screens

Fine screens should not be considered equivalent to primary sedimentation but may be used in lieu of primary sedimentation where subsequent treatment units are designed on the basis of anticipated screen performance. Selection of screen capacity should consider flow restriction due to retained solids, gummy materials, frequency of cleaning and extent of cleaning. Where fine screens are used, additional provision for removal of floatable oils and greases should be considered. Care should be taken with smaller screen size openings to avoid plugging.

Air exchanges similar to coarse screens need to be provided (*Section 10.1.1.2 - Access and Ventilation*).

If there is a high organic content in the screenings, washing is an effective way of breaking up and reducing the amount of fecal and organic content. This will help to reduce odours within the screening area and reduce the solid content for disposal. Washers are most efficient when used in combination with compactors, since water is added to break-up organics and this water should be removed.

Volume reduction is a means to minimize the cost of disposal. Depending on the characteristics of the screenings, they can be effectively dried to 50% moisture content and reduced by up to 75% of their original volume, reducing hauling and disposal cost. Screw compactors and piston-type compactors are used to dewater and compact screenings. Compacting is an approach to save costs and in some cases disposal sites require screenings to pass specific standards for dryness.

10.1.2.1 Design and Installation

Tests should be conducted to determine BOD_5 and suspended solids removal efficiencies at the design maximum day flow and design maximum day BOD_5 loadings. Pilot testing for an extended time is preferred to cover key seasonal operational variations.

It is recommended that a minimum of two fine screens be provided, each unit being capable of independent operation. Capacity is recommended to be provided to treat design peak instantaneous flow with one unit out of service.

Fine screens should be preceded by a coarse bar screening device. Fine screens should be protected from freezing and located to facilitate maintenance.

It is recommended that hosing equipment be provided to facilitate cleaning. Provisions should be made for isolating and removing units from their location for servicing.

10.1.3 Microscreening

Microscreens are classified as having less than 1 mm (1/25 in) screen openings. Screens can be constructed of different types of material such as woven metal, perforated metal plates and woven cloth. This screen category is conventionally used as a polishing step, although recently some companies have explored the possibilities of providing complete pretreatment to create a more compact STP and to replace the need for primary treatment.

Similar to fine screens, microscreening may be accomplished directly or indirectly. The effectiveness of the direct method of capturing solids is largely dependent on the size of the screen openings. Indirect capture of solids will occur when a mat or film develops on the screen from previous solids retention. This will reduce the effective size of the screen opening and hence, increase the overall efficiency of the screening process. Caution should be used when indirect filtration occurs with microscreens since there is a high potential for fouling and excess headloss.

10.2 COMMINUTORS AND GRINDERS

10.2.1 General

Grinders and comminutors (including barminutors) represent an alternative to coarse screening. Since problems can occur due to the recombining of comminuted or barminuted materials in downstream treatment units, it is recommended that the physical removal of the coarse material from the sewage influent be used rather than the use of shredding or cutting devices with reintroduction of the material to the treated sewage.

When comminutors are used, they are commonly placed downstream of grit removal to avoid damage to the cutters caused by grit particles. On the other hand, if comminution is provided upstream of grit removal units, more efficient grit removal will be achieved. If mechanical grit removal is used, equipment protection in the form of some type of coarse screening device should be provided upstream of the grit removal facilities.

10.2.2 Design Considerations

Comminutors should be protected by a coarse screening device. Comminutors not preceded by grit removal equipment are recommended to be protected by a 150 mm (6 in) deep gravel trap.

Comminutors may be used in lieu of screening devices to protect equipment where stringy substance accumulation on downstream equipment will not be a substantial problem.

It is recommended that comminutor capacity be adequate to handle design peak hourly flow. A screened bypass channel should be provided and should be automatic for all comminutor failures. Channel gates should be provided where necessary.

Provision needs to be made to facilitate servicing units in place and for removing units from their location for servicing. Provisions for access, ventilation, shields and safety should be in accordance with Sections 10.1.1.2 and 10.1.1.3.

Electrical equipment in comminutor chambers where hazardous gases may accumulate needs to meet the requirements of the *Electrical Safety Code for Class I, Zone 1,* Group D locations (O.Reg. 164/99 made under the *Electricity Act, 1998*). Motors need to be protected against accidental submergence.

10.3 GRIT REMOVAL FACILITIES

10.3.1 General

Grit removal facilities should be provided for all mechanical STPs especially those receiving sewage from combined sewers or from sewer systems receiving substantial amounts of grit. If a plant serving a separate sewer system is designed without grit removal facilities, it is recommended that the design include provision for future installation. It is also recommended that consideration be given to the possible damaging effects on pumps, comminutors and other preceding as well as downstream equipment and the need for additional storage capacity in treatment units where grit is likely to accumulate.

The quantity of grit removed at an STP can vary significantly depending on the sewage flow, characteristics of the service area, type of collection system and type of screen located before grit collection. Grit collection can vary from 4 to 37 mL/m^3 of treated sewage (0.54 to 4.9 cu ft/mil. US gal) for separate sewer systems and 4 to 180 mL/m^3 (0.54 to 24.1 cu ft/mil. US gal) for combined sewer systems.

Grit removal is normally accomplished by grit channels, detritus tanks, aerated grit tanks or vortex grit tanks. Automated grit removal equipment is preferred to avoid manual cleaning (i.e., as required with grit channels). Grit removal can also be accomplished using centrifugal type separators and stationary screens, although these are less commonly used in Ontario.

10.3.2 Design Factors

Grit removal facilities should be located ahead of pumps and comminuting devices. Coarse bar racks should be placed ahead of grit removal facilities. Grit removal facilities located outside should be protected from freezing. Heat tracing for example may be required on specific equipment or processes. It is recommended that adequate stairway access to above- or below-grade facilities be provided.

Ventilation should be provided, with recommended continuous fresh air introduction rates of at least 12 air changes per hour, or intermittently at a rate of at least 30 air changes per hour. Odour control facilities may also be warranted.

All electrical work in enclosed grit removal areas where hazardous gases may accumulate needs to meet the requirements of the *Electrical Safety Code for Class I, Zone 1*, Group D locations (O.Reg. 164/99 made under the *Electricity Act, 1998*). Explosion proof gas detectors need to be provided.

Plants treating sewage from combined sewers should have at least two mechanically cleaned grit removal units, with provisions for bypassing. A single manually cleaned or mechanically cleaned grit chamber with bypass is acceptable for small sewage treatment plants serving separate sanitary sewer systems. Minimum facilities for larger plants serving separate sanitary sewers should be at least one mechanically cleaned unit with a bypass.

Facilities other than channel-type should be provided with adequate and flexible controls for velocity and/or air supply devices and with grit collection and removal equipment. Aerated grit tanks should have air rates adjustable in the range of 4.7 to 12.4 L/(m·s) (3 to 8 cfm/ft of tank length). Detention time in the tank should be in the range of 3 to 5 minutes at the design peak hourly flow.

The design effectiveness of a grit removal system should be commensurate with the requirements of the subsequent process units.

Inlet turbulence should be minimized in channel type units. Channel-type chambers should be designed to control the velocities during normal variations in flow as close as possible to 0.3 m/s (1 ft/s). The detention period should be based on the size of particle to be removed.

All aerated grit removal facilities should be provided with adequate control devices to regulate air supply and agitation.

The need for grit washing should be determined by the method of grit handling and final disposal.

The designer should make provision for isolating and draining each unit. It is recommended that the design provide for complete draining and cleaning by means of a sloped bottom equipped with a drain sump. An adequate supply of water under pressure should be provided for cleanup.

Grit removal facilities located in deep pits should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage are recommended for grit handling areas. Grit transporting facilities should be provided with protection against freezing and loss of material. Hoisting equipment needs to be provided or available that is capable to lift all mechanical equipment for servicing and repair.

10.3.3 Types of Grit Removal Systems

10.3.3.1 Grit Channels

Grit channels are usually employed in smaller plants. Grit removal is accomplished by velocity control provided by proportional weirs. Grit channels are normally manually cleaned. The design parameters for grit channels are as follows:

- Number of channels – minimum of 1; recommend at least 2 for larger sewage treatment plants (with one channel out of service there should be enough capacity in remaining units to handle the design peak hourly flow);
- Control velocity - 0.3 m/s (1 ft/s);
- Control weirs - proportional, Sutro (or Parshall in parabolic channels);
- Minimum channel width - 380 mm (15 in);
- Minimum length - that required to settle 0.2 mm (1/16 in) particle with a specific gravity (SG) of 2.65 plus 50 per cent allowance for inlet and outlet turbulence. However, analysis of grit removal data indicates that the SG ranges from 1.3 to 2.7; and
- Grit storage - with permanently positioned weirs, the weir crest should be kept 150 to 300 mm (6 to 12 in) above the grit channel invert to provide for storage of settled grit (weir plates that are capable of vertical adjustment are preferred since they can be moved to prevent the sedimentation of organic solids following grit cleaning).

10.3.3.2 Detritus Tanks

Detritus tanks should be designed with sufficient surface area to remove the same, or smaller, particle size and density as required for grit channels at the design peak hourly flow rate. Detritus tanks, since they are mechanically cleaned and do not need dewatering for cleaning, do not require multiple units.

The grit settled in the detritus tank will have a significant organic content due to the lighter solids settling out during low flow periods. Separation of the organics from the grit before, during, or after the removal of the settled contents of the tank should accomplish in one of the following ways:

- Compressed air can be diffused into the tank periodically to re-suspend organic material;

- The removed detritus can be washed in a grit washer with the organic laden wash water being returned to the head of the detritus tank;
- A classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank; and
- The removed detritus can be passed through a centrifugal-type separator.

10.3.3.3 Aerated Grit Tanks

Aerated grit tanks for the removal of 0.2 mm (1/16 in), or larger, particles with specific gravity of 2.65 should be designed in accordance with the following parameters:

- Detention time - 2 to 5 minutes at design peak hourly flow rate (the longer retention times provide additional benefit in the form of preaeration);
- Air supply - 4.7 to 12.4 L/(m·s) (3 to 8 cfm/ft), via wide band diffusion header positioned lengthwise along one wall of tank; (air supply should be variable);
- Inlet conditions - inlet flow should be parallel to induced roll pattern developed in tank;
- Baffling - minimum of one transverse baffle near outlet weir, with additional transverse baffles in long tanks and longitudinal baffles in wide tanks;
- Outlet conditions - outlet weir oriented parallel to direction of induced roll;
- Tank dimensions - lower limit of above aeration rates generally suitable for tanks up to 3.7 m (12 ft) deep and 4.3 m (14 ft) wide; wider, or deeper tanks require aeration rates in the upper end of the above range; long, narrow aerated grit tanks are generally more efficient than short tanks and produce cleaner grit; length-to-width (L/W) ratio normally is 1.5:1 to 2:1, but up to 5:1 is acceptable; depth-to-width (D/W) ratio 1:1.5 to 1:2;
- Desired velocities - surface velocity in the direction of roll in tanks should be 0.45 to 0.6 m/s (1.5 to 2.0 ft/s) (tank floor velocities will be approximately 75 per cent of above);
- Grit collectors - air lifts, pumps, mechanical conveyors or clam shell buckets may be used for the removal of grit (pretreatment in the form of screening will be required upstream of mechanical grit removal processes);
- Grit washing - depending upon the method of removal and ultimate disposal, the grit may have to be washed after removal by devices of the type discussed in the previous section;

- Multiple units - generally not required, or where grit removal method requires bypassing of tank (as with clam shell bucket); and
- Tank geometry - critical with respect to location of air diffusion header, sloping tank bottom, grit hopper and fitting of grit collector mechanism into the tank structure. Consultation with equipment suppliers is advisable.

10.3.3.4 Vortex Grit Removal

The vortex grit removal systems are proprietary and rely on a mechanically induced vortex to capture grit solids in the center hopper of a circular tank. The designer should ensure that the manufacturer verifies that the appropriately sized unit has been field tested to determine performance parameters and should consider its performance during low flow periods. The designer should obtain design data from the manufacturer for appropriate entrance and exit channels and a concrete tank for installation of the grit removal equipment.

10.4 PREAERATION

Preaeration of sewage to reduce septicity may be required in special cases. Preaeration can be incorporated through extended use of aerated grit process or by the provision of a separate unit process.

10.4.1 Mixing

Aeration or mechanical equipment should be provided to maintain adequate mixing. Corner fillets and hopper bottoms with draw-offs should be provided to alleviate the accumulation of sludge and grit. A spray wash down system or tipping bucket should be included to wash down tank after use.

10.4.2 Aeration

Aeration equipment should be sufficient to maintain a minimum of 1.0 mg/L of dissolved oxygen in the mixed basin contents at all times. Air supply rates should be a minimum of 0.16 L/(m^3·s) (1.25 cfm/1000 US gal of storage capacity). The air supply should be isolated from other treatment plant aeration requirements to facilitate process aeration control, although process air supply equipment may be utilized as a source of standby aeration. Although coarse bubble diffusers have been used in the past, consideration for using fine pore diffusers should be made due to the higher oxygen transfer efficiency and current reliability (e.g. membrane diffusers).

10.4.3 Controls

Inlets and outlets for all basin compartments should be suitably equipped with accessible external valves, stop plates, weirs, or other devices to permit flow control and the removal of an individual unit from service. Equipment should be provided to measure and indicate liquid levels and flow rates.

10.4.4 Access

Suitable access should be provided to facilitate cleaning and the maintenance of equipment.

10.5 FLOW EQUALIZATION

Full equalization of diurnal sewage flow peaks can result in a reduction in construction costs over variable flow design and can also result in reduced energy costs and improved treatment efficiency. Partial or side-line equalization minimizes pumping requirements but is less effective at equalizing pollutant concentrations. Consideration should be made for taking any tanks out of service for cleaning or maintenance either by multiple tanks or provision to bypass.

10.5.1 General

Use of flow equalization should be considered where significant variations in organic and hydraulic loadings can be expected.

10.5.2 Location

Equalization basins should be located downstream of pretreatment facilities such as bar screens, comminutors and grit chambers.

10.5.3 Type

Flow equalization can be provided by using separate basins or on-line treatment units, such as aeration tanks. Equalization basins may be designed as either in-line or side-line units. Unused treatment units, such as sedimentation or aeration tanks, may be utilized as equalization basins during the early period of design life.

10.5.4 Size

Equalization basin capacity should be sufficient to effectively reduce expected flow and load variations. With a diurnal flow pattern, the volume required to achieve the desired degree of equalization can be determined from a cumulative flow plot over a representative 24-hour period.

10.5.5 Electrical

All electrical work in housed equalization basins, where hazardous concentrations of flammable gases or vapours may accumulate, needs to meet the requirements of the *Electrical Safety Code for Class I, Zone 1,* Group D locations (O.Reg. 164/99 made under the *Electricity Act, 1998*).

10.6 SCREENINGS, GRIT HANDLING AND DISPOSAL

Special consideration needs to be given to the design of screenings and grit handling systems to ensure the material is easy to handle, odours are reduced and acceptable for final disposal.

The design of screenings handling equipment will be also dictated by disposal practices. Landfill practices are changing and some landfills do no accept material containing free water or fecal material. Screenings disposed of through a transfer station may require additional considerations. Screenings handling devices include:

- **Belts and Dumpsters** - screenings may be moved to a dumpster by belts. The belts will need to be cleaned, so a nearby wash station should be included in the design. Because screenings in the dumpster will generate odours and attract insects, enclosing the dumpster should be considered;
- **Washers** - screenings from screens with smaller openings (i.e., less than 12 mm or 0.5 in) will contain fecal material. Washers should be considered that will remove fecal material from the screenings. Most washers are combined with compactors that remove excess water from the rags; and
- **Compactors** - compactors, when used with screenings, will remove excess water so that landfills will accept the waste. If the compactor is placed outside, the discharge tube should be heat-taped and insulated. Large amounts of rock in screenings will cause binding problems in the discharge tube. Flushing or an alternative means of dewatering should be considered.

Most screenings storage will produce odours, vectors problems and drainage. Odour control and proper ventilation should be addressed in all storage container siting decisions.

Dumpsters that receive screenings should have a way to be dewatered with a floor drain to the sanitary sewer, as close as possible to the dumpster. Drainage from dumpsters may damage concrete floors because of acidity, so the floor should have a protective coating. A cleanup station should be in the immediate area for cleaning when the dumpster is picked up. Redundancy or another method of screenings handling should be considered in case of equipment failure. Because screenings and storage rooms have corrosive atmospheres, all equipment should be of noncorrosive design.

Grit washing effectively removes organics from the grit. Screw and rake grit washers have proved to be reliable and usually produce a material low in organics. To ensure a low volatile content, ample dilution water may be required. Pumps normally provide sufficient dilution water, but bucket elevators may not, especially during periods of peak grit capture. Consequently, they may require supplementary liquid to function properly.

Disposal of screenings and grit is the most critical design consideration. Most landfills cannot accept waste that contains free water. Some will not accept waste with visible fecal material. The design of the dumpster box and the type of screenings/grit handling will be dictated, in most cases, by these landfill requirements.

CHAPTER 11

PRIMARY SEDIMENTATION

CHAPTER 11

PRIMARY SEDIMENTATION

This chapter describes the primary sedimentation process that typically follows the preliminary treatment process of most municipal sewage treatment plants. Design considerations and descriptions of different types of primary sedimentation tanks (also known as primary clarifiers), as well as primary sludge and scum collection and removal systems are included in this chapter. A summary of the design loadings for primary clarifiers is provided in Appendix V which should be used in conjunction with the details in this chapter.

11.1 GENERAL

The need for primary sedimentation tanks may be governed by the need to remove scum and grease or other debris prior to secondary treatment. The designer of primary sedimentation tanks should consider the following factors:

- The characteristics of the raw sewage;
- The type of sludge digestion system, either available or proposed (aerobic digestion should not be used with raw primary sludge);
- The type of secondary treatment following primary treatment;
- The need for handling of waste activated sludge in the primary sedimentation tank(s); and
- The need for phosphorus removal in the primary sedimentation tank(s).

Primary sedimentation provides low-cost suspended solids and BOD_5 removal, especially in cases where the raw sewage contains a high proportion of settleable solids, as is often the case with sewage containing significant food processing waste, or similar wastes. Primary sedimentation may also incorporate ballasting or recirculation to enhance solids settling.

Primary sedimentation tanks used for phosphorus precipitation with normal strength municipal sewage typically exhibit BOD_5 and suspended solids removals of 45 and 85%, respectively. Without chemical addition for phosphorus removal, the BOD_5 and suspended solids reductions typically would be 35 and 65%, respectively. For suspended solids removal, the range is 60% to 90% with chemical addition and 40% to 70% without. Actual removal rates depend mainly on raw sewage characteristics and contributing sources (e.g. industrial inputs), chemical dosage (if any), mixing and clarifier hydraulics. BOD_5 removal rates are affected by the proportion of soluble to particulate fractions of BOD_5 in the raw sewage. The use of the secondary clarifiers for phosphorus removal has been the most common approach. This has been at least partially due to the reduced chemical requirements when the secondary units are used for phosphorus removal.

In view of the potential for increased BOD_5 and suspended solids removals, there may be circumstances when consideration should be given to the use of primary sedimentation tanks for phosphorus removal rather than the secondary sedimentation tanks. Such circumstances might include the following:

- Where existing aeration tanks and/or secondary clarifiers are overloaded;
- Where nitrification is made a requirement for an existing secondary treatment plant;
- Where excessive waste activated sludge production is causing anaerobic digester operating problems; and
- Where economic evaluation shows the process to be more cost effective despite the higher chemical requirement and costs.

The use of the primary sedimentation tanks for phosphorus removal will generally permit removal down to the 1.0 mg/L level. However, if lower phosphorus levels are required, chemical addition to the primary sedimentation tanks may not be successful. This problem is at least partially due to the fact that some forms of phosphorus are more amenable to precipitation after aeration and that the phosphorus level variations are generally greater in raw sewage than experienced in the aeration tank effluent. It is therefore recommended that precipitation testing (e.g. jar testing) be carried out before a final decision is made on which plant treatment units are to be used for phosphorus removal.

Primary sedimentation tanks can be either rectangular or circular. With rectangular tanks, length to width (L/W) ratios of at least 4:1 are preferred. Width to depth (W/D) ratios of 1:1 to 2.25:1 are typical.

Factors to be considered in the selection of either rectangular or circular primary sedimentation tanks are outlined below.

11.1.1 Rectangular Tanks

- Permit common wall construction;
- Usually result in a thicker sludge;
- Usually less expensive to cover, if chain and flight-type collector is used;
- Traveling bridge type collectors may be less expensive than rotary circular collectors for large tanks, although chain and flight type collectors are often more popular and allow easier covering of tank if warranted (e.g. for odour control); and
- Width of tank controlled by sludge withdrawal mechanism.

11.1.2 Circular Tanks

- Rotary circular sludge collector mechanism usually less costly for small tanks and requires less maintenance than chain and flight type collectors for rectangular tanks;
- Potential for pre-cast construction;
- Sludge sump can be equipped with a blade to provide stirring to avoid sludge bridging;
- Usually more susceptible to short-circuiting; and
- For tank depths greater than 3 m (10 ft), may be less expensive than rectangular tanks.

11.2 DESIGN CONSIDERATIONS

11.2.1 Number of Units

Multiple primary sedimentation tanks capable of independent operation are desirable and should be provided in all plants where design average daily flows exceed 380 m^3/d (0.1 mUSgd). Plants not having multiple units should include other provisions to assure continuity of treatment to meet the final effluent quality criteria.

11.2.2 Flow Distribution

Effective flow splitting devices and control appurtenances (i.e., gates and splitter boxes) need to be provided to permit proper proportioning of flow and solids loading to each unit, throughout the expected range of flows. (*Section 3.13.2 - Flow Distribution* and *Section 8.5.8 - Flows and Organic Loadings Distribution*.)

11.2.3 Dimensions

It is recommended that the minimum length from the inlet to the outlet be 3 m (10 ft) unless special provisions are made to prevent flow short-circuiting. The side water depth (SWD) should be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs. Generally, primary sedimentation tanks have a SWD from 3.0 to 4.6 m (10 to 15 ft).

11.2.4 Surface Overflow Rates

Primary sedimentation tank sizing should reflect the degree of solids removal required and the need to avoid septic conditions during low flow periods. The primary sedimentation tanks surface overflow rates (SOR) are shown in Table 11-1. It is recommended that sizing be calculated for both design average daily flow and design peak daily flow conditions and the larger surface area determined to be used.

Table 11-1 shows the recommended design parameters for primary sedimentation tanks. The recommended SOR should be used for design unless the designer can demonstrate that higher SOR can be accommodated and still achieve the required treatment efficiency. Required treatment efficiency would be based on expected removal efficiency and the capacities of downstream processes. For instance, for plant expansions, it may be possible to show through full-scale testing of the existing primary treatment units that higher SOR will produce the desired results. Although not a common design parameter, the hydraulic retention time in a primary sedimentation tank generally varies from 1.5 to 2.5 hours at design average daily flow. Two hours is a typical value.

Table 11-1- Primary Sedimentation Tanks Surface Overflow Rates

Type of Primary Sedimentation Tank[2]	Surface Overflow Rates at:[1]	
	Design Average Daily Flow $m^3/(m^2 \cdot d)$ (USgpd/ft^2)	Design Peak Daily Flow $m^3/(m^2 \cdot d)$ (USgpd/ft^2)
Tanks not receiving waste activated sludge [3, 4]	30 – 40 (740 - 980)	60 - 80 (1470 – 1960)
Tanks receiving waste activated sludge[4]	25 – 30 (610 – 740)	50 - 60 (1230 – 1470)

Notes:

1. Surface overflow rates need to be calculated with all flows received at the primary sedimentation tanks. Primary settling of normal domestic sewage can be expected to remove approximately 1/3 of the influent BOD_5 when operating at an overflow rate of 40 $m^3/(m^2 \cdot d)$ (980 USgpd/ft^2).
2. The following design SOR has traditionally been used in the past for the design of primary treatment plants, that is at design average and peak daily flows of < 35 (860) and < 70 (1720) $m^3/(m^2 \cdot d)$ (USgpd/ft^2).
3. Anticipated BOD_5 removal should be determined by laboratory tests and consideration of the character of the wastes. Significant reduction in BOD_5 removal efficiency may result when the peak daily overflow rate exceeds 60 $m^3/(m^2 \cdot d)$ (1470 USgpd/ft^2).
4. Waste activated sludge in this instance would also include biological waste sludge from other biological processes including fixed film systems.

11.2.5 Inlet Structures

Inlets and baffling should be designed to dissipate the inlet velocity, to distribute the flow equally both horizontally and vertically, to protect the sludge hopper and to prevent short-circuiting. It is recommended that channels be designed to maintain a velocity of at least 0.3 m/s (1 ft/s) at one-half of the design average daily flow. It is recommended that corner pockets and dead zones be eliminated and corner fillets or channeling be used where necessary. Provisions should be made for elimination or removal of floating materials which may accumulate in inlet structures.

11.2.6 Weirs

11.2.6.1 General

Overflow weirs should be readily adjustable over the life of the structure to correct for differential settlement of the tank.

11.2.6.2 Location

Overflow weirs should be located to optimize actual hydraulic detention time and minimize short-circuiting. It is recommended that peripheral weirs be placed at least 0.3 m (1 ft) from the wall.

11.2.6.3 Design Rates

It is recommended that weir loadings not exceed values shown in Table 11-2.

Table 11-2– Recommended Weir Loading Rates

Average Plant Capacity	Loading Rate at Design Peak Daily Flow $m^3/(m \cdot d)$ (USgpd/ft)
Equal to or less than 4000 m^3/d (1 mUSgd)	250 (20,000)
Greater than 4000 m^3/d (1 mUSgd)	375 (30,000)

If influent pumping is required, the pumps should be operated as continuously as possible. Also, weir loadings should be related to pump delivery rates to avoid short-circuiting during pump operations, although peak flow conditions are expected to govern.

11.2.6.4 Weir Troughs

Weir troughs should be designed to prevent submergence at design peak hourly flow and to maintain a velocity of at least 0.3 m/s (1 ft/s) at one-half design average daily flow.

11.2.7 Submerged Surface

The tops of troughs, beams and similar submerged construction elements should have a minimum slope of 1.4 vertical to 1 horizontal; the underside of such elements should have a slope of 1 to 1 to prevent the accumulation of scum and solids.

11.2.8 Sedimentation Tank Dewatering

The ability to dewater a primary sedimentation tank and take tanks out of service should conform to the provisions outlined in *Section 8.4.15 - Component Backup Requirements*. It is recommended that primary sedimentation feed channels be designed to provide for distribution of peak sewage flow to the remaining tanks, when one tank is out of service and/or dewatered. Consideration should be given to provide adequate means for dewatering of tanks, for example a sump of adequate size for temporary insertion of a submersible pump to dewater the tank.

11.2.9 Freeboard

It is recommended that the walls of primary sedimentation tanks extend at least 150 mm (6 in) above the surrounding ground surface and provide not less than 300 mm (12 in) freeboard. Additional freeboard or the use of wind screens is recommended where larger primary sedimentation tanks are subject to high velocity wind currents that would cause tank surface waves and inhibit effective scum removal.

11.3 SLUDGE AND SCUM REMOVAL

11.3.1 Scum Removal

Full-surface mechanical scum collection and removal facilities, including baffling, should be provided for all primary sedimentation tanks. The characteristics of scum, which may adversely affect pumping, piping, sludge handling and disposal, need to be recognized in design. Scum pits may require heating to avoid freezing problems and consideration should be given to mixers. Smooth-walled pipe should be used for scum lines to minimize grease buildup. Glass-lined pipe is recommended for scum piping. Scum lines should be provided with clean outs or steam injection points to minimize blockage due to grease buildup.

Provisions should be made to remove scum from the sewage treatment process and direct it to either the sludge treatment process or an alternative treatment and disposal process. Scum treatment can be provided by digestion, but this can lead to problems in the digesters. If treated by digestion, consideration should be given to injecting the scum into the sludge recirculation line downstream of heat exchanger. As an alternative approach, scum may be transferred directly to landfill with screenings or to dewatering or incineration units, if available. Other special provisions for disposal may be necessary.

11.3.2 Sludge Removal

Mechanical sludge collection and withdrawal facilities should be designed to ensure rapid removal of sludge.

Each sedimentation tank should have its own sludge withdrawal line to ensure adequate control of sludge removal rate for each tank. Sludge removal needs to be adequate to handle the expected maximum sludge accumulation rate and avoid excessive blanket levels.

11.3.2.1 Sludge Hopper

The minimum slope of the side walls should be 1.7 vertical to 1 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms should have a maximum dimension of 0.6 m

(2 ft). Extra depth sludge hoppers for sludge thickening can be considered but should be designed appropriately.

11.3.2.2 Cross-Collectors

Cross-collectors serving one or more primary sedimentation tanks may be considered in place of multiple sludge hoppers.

11.3.2.3 Sludge Removal Pipeline

Each hopper should have an individually valved sludge withdrawal line at least 150 mm (6 in) in diameter, although the need to maintain minimum pipe velocities and pump run times may require consideration of smaller diameter pipes. The static head available for withdrawal of sludge should be 760 mm (30 in) or greater, as necessary to maintain a 0.9 m/s (3 ft/s) velocity in the withdrawal pipe. Clearance between the end of the withdrawal line and the hopper walls should be sufficient to prevent bridging of the sludge. Adequate provisions should be made for rodding or back-flushing individual pipe runs. Piping should be provided to remove sludge for further processing.

11.3.2.4 Sludge Removal Control

Separate primary sedimentation tank sludge lines may drain to a common sludge well. Sludge wells equipped with telescoping valves or other appropriate equipment should be provided for viewing, sampling and controlling the rate of sludge withdrawal. A means of measuring the sludge removal rate should be provided. Air-lift pumps are not recommended for the removal of primary sludge.

11.4 SAFETY

All primary sedimentation tanks should be suitably equipped to enhance safety for operators and include machinery covers, life lines, stairways, walkways, handrails and slip resistant surfaces. The design should provide for convenient and safe access to routine maintenance items such as gear boxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle areas and effluent channels. Electrical equipment, fixtures and controls in enclosed settling basins and scum tanks, where hazardous concentrations of flammable gases or vapors may accumulate, should comply with the requirements contained in *Section 8.9 - Safety*. The explosion-proof classification may need to extend to equipment within a close envelope to an open tank. The fixtures and controls should be located so as to provide convenient and safe access for operation and maintenance. Adequate area lighting should be provided.

CHAPTER 12

BIOLOGICAL TREATMENT

CHAPTER 12

BIOLOGICAL TREATMENT

This chapter describes biological treatment processes including design, construction and operational considerations. Suspended growth systems using the activated sludge process with its variations, lagoons, and fixed-film systems are described in this chapter. A summary of the design loadings for conventional biological processes is provided in Appendix V, which should be used in conjunction with the details in this chapter.

12.1 PROCESS SELECTION

The activated sludge process (ASP) and its variations (including the sequencing batch reactor process) is the most common secondary treatment process used in Ontario. Other processes including fixed-film treatment systems are also capable of meeting secondary effluent quality (15 mg/L $CBOD_5$, 15 mg/L total suspended solids).

A list of the common biological processes including suspended growth, fixed film, and hybrid (combined suspended and fixed-film) systems that are well known and proven technologies for use in North America is provided below:

- Suspended Growth Processes:
  - Conventional Activated Sludge (CAS) process;
    - Plug Flow;
    - Complete Mix;
    - Contact Stabilization;
    - Extended Aeration;
    - Step-Feed ASP; and
    - High-Rate ASP.
  - Membrane Bioreactor (MBR);
  - Sequencing Batch Reactor (SBR); and
  - Lagoon (Facultative and/or Aerated).
- Fixed Film Processes:
  - Rotating Biological Contactor (RBC); and
  - Trickling Filter (TF).
- Hybrid Processes:
  - Integrated Fixed-film Activated Sludge (IFAS);
  - Trickling Filter/Solids Contact (TF/SC);
  - Rotating Biological Contactor/Solids Contactor (RBC/SC); and
  - Biological Aerated Filter (BAF).

12.2 ACTIVATED SLUDGE PROCESS

12.2.1 General

The activated sludge technology and its several variations rely on aeration tanks for biological treatment and a means of sludge retention within the process. The ASP may be utilized to accomplish varied degrees of removal of total suspended solids and reduction of carbonaceous and/or nitrogenous oxygen demand. The major ASP types include: plug-flow, complete-mix, high-rate, contact stabilization, extended aeration and step-feed systems. Choice of the most appropriate process type or mode of operation should depend on the effluent quality criteria, consistency of treatment required, characteristics of sewage, proposed plant size, anticipated degree of operation with maintenance requirements, operating and capital costs. All designs should provide for flexibility in operation and, if feasible, should allow for operation in various modes.

The activated sludge process requires close operational attention and control, including routine monitoring and laboratory analyses. These requirements, which are relatively independent of plant size, should be considered when proposing this type of treatment.

The activated sludge process requires considerable amounts of energy to satisfy aeration demands. The energy demand is often 40 to 60 percent of the total energy usage for the overall sewage treatment plant (STP). Capability of energy usage reductions while still maintaining viability of the treatment process, both under normal and emergency conditions, should be included in the ASP design.

Protection against low temperatures and excessive heat loss should be provided to ensure continuity of operation and performance. Insulation of the tanks by earthen banks should be considered.

12.2.2 Pretreatment

Effective removal or exclusion of grit, debris, excessive oil or grease and screening of solids should be accomplished prior to the activated sludge process. Description of the facilities that may need to be provided is included in *Chapter 10 - Preliminary Treatment*.

Where primary treatment is used, provision should be made for discharging raw sewage (after screening and grit removal) directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life and to provide operational flexibility.

12.2.3 Selectors

Selectors can be used to enhance the selection of desired organisms in the ASP, to reduce the growth of filamentous organisms and to enhance the settling of the mixed liquor suspended solids. Selectors can be aerobic, anoxic or anaerobic. All selectors should be designed to provide a substrate concentration gradient, a high initial *Food-to-Microorganism* (F/M) ratio and adequate time for the absorption of soluble organic material by the microorganisms. The F/M ratios indicated in sections 12.2.3.1 through 12.2.3.3 are calculated as follows:

F = Influent BOD_5 (g/m^3) · Influent flow rate (m^3/d)

M = MLVSS in the tank(s) (g/m^3) · Volume of tank(s) (m^3)

where:

MLVSS = Mixed Liquor Volatile Suspended Solids concentration in the tank(s) (g/m^3)

F = BOD_5 mass loading rate to first selector compartment. The same F is used in the F/M calculations for the other compartments as well.

M = Total mass of MLVSS in the compartments considered. Mass for the first compartment is MLVSS concentration multiplied by the first compartment volume. For the second compartment, the MLVSS concentration is multiplied by the combined volume of the first and second reactors and for the third compartment, MLVSS concentration is multiplied by the volume of all three compartments.

12.2.3.1 Aerobic Selectors

Three-compartments should be used. The F/M ratio of the first compartment is critical. The following F/M gradient, which would result in two equally sized compartments followed by a third compartment with twice the volume of the first compartment, is recommended:

- 1st compartment 24 d^{-1}
- 2nd compartment 12 d^{-1}
- 3rd compartment 6 d^{-1}

Aerobic selectors should maintain 1 to 2 mg/L of dissolved oxygen (DO). Provision should be made to satisfy oxygen uptake rates (OUR) of at least 65 to 80 mg of oxygen/g of MLVSS/hr [65 to 80 lb/1000 lb of MLVSS/hr].

12.2.3.2 Anoxic Selectors

Anoxic selectors are used in nitrifying activated sludge systems. A portion of the nitrified mixed liquor is recycled to the anoxic zone for denitrification. Sufficient nitrate-N concentration needs to be maintained in the recycled stream to the anoxic zone to remove the soluble BOD_5 and maintain anoxic conditions. A single-stage selector design with an F/M of 0.5 to 1 d^{-1} will generally be effective for filamentous control.

More efficient selection may be achieved with a three-compartment anoxic configuration. The following F/M gradient is recommended:

- 1st compartment $12\ d^{-1}$
- 2nd compartment $6\ d^{-1}$
- 3rd compartment $3\ d^{-1}$

The anoxic zone or components should be mixed with mechanical mixers or by very low aeration rates. If low aeration is used for mixing, the DO should be kept below 0.5 mg/L.

12.2.3.3 Anaerobic Selectors

The anaerobic selector retention time should be 0.75 to 2.0 hours. The zone may be divided into three compartments with similar F/M ratios as the anoxic zone. Dissolved oxygen and nitrate cannot be present for the selector to act as an anaerobic selector. Mechanical mixers need to be used to maintain the solids in suspension in anaerobic selectors.

12.2.4 Aeration

12.2.4.1 General

The designer of aeration tanks and associated equipment should consider the following:

- Expected oxygen demands, including variations, exerted by sewage flows from upstream treatment units;
- Hydraulic loading rates, including variability;
- Treatment requirements, including reduction of $CBOD_5$, and nitrification if necessary;
- Temperature, pressure, relative rate of oxygen transfer (Alpha factor) and relative oxygen saturation values (Beta factor) for the sewage; and
- Other factors including type of recycle streams, aeration equipment and surfactants in the sewage.

The design parameters for the aeration systems associated with various activated sludge treatment processes are given in Table 12-1. These design parameters are applicable to both complete-mixed and plug-flow systems.

12.2.4.2 Aeration System Alternatives

Both mechanical and diffused aeration systems should be considered including standby equipment needs.

The designer should evaluate the aeration system alternatives considering the following factors:

- Oxygen transfer efficiencies;
- Power requirements;
- Diffuser clogging problems;
- Mixing capabilities;
- Air pretreatment requirements;
- Aerator tip speed of mechanical aerators;
- Icing problems;
- Misting problems;
- Cooling effects on aeration tank contents;
- Estimated installed capital cost;
- Expected maintenance costs; and
- Estimated operating costs.

12.2.4.3 Oxygen Transfer Efficiencies and Rates

The typical process oxygen transfer efficiencies and rates for water at 0 mg/L DO, 20 °C and 101 kPa (1 atm) atmospheric pressure, commonly used for aeration devices are given below:

- Coarse bubble diffusers - 4 to 6 percent [based on average tank depth of 4.5 m (15 ft)];
- Fine bubble diffusers - 6 to 15 percent [based on average tank depth of 4.5 m (15 ft)];
- Low speed mechanical aerators (70 rpm or less) - 1.5 to 2.7 kgO_2/kWh [2.5 to 4.4 lb/(hp·hr)];
- Submerged turbines - 1.0 to 1.5 kgO_2/kWh [1.6 to 2.5 lb/(hp·hr)];
- High-speed mechanical aerators - 1.2 to 1.5 kgO_2/kWh [2.0 to 2.5 lb/(hp·hr)]; and
- Brush rotors - 1.5 to 2.1 kgO_2/kWh [2.5 to 3.5 lb/(hp·hr)].

Higher oxygen transfer efficiencies and rates than stated above may be considered if the designer or equipment supplier can document or show through pilot-scale or full-scale testing that higher rates can be achieved.

There are also other aeration methods such as pure oxygen addition systems, jet aerators, tubular aerators which may be considered.

Table 12-1 - Aeration System Design Parameters[11, 13]

Treatment Process	Organic Loading Rate[12] (kg BOD_5 / ($m^3 \cdot d$))	F / M_v[1] (d^{-1})	Minimum Retention Time (Based on Q Avg.)	Return Sludge Rate[2] (% Q Avg.)	Oxygen Demand in Typical Municipal Sewage at Standard Conditions[9]	Solids Retention Time (SRT) (Days)	MLSS Concentration (mg/L)
Conventional A.S. [(3)]							
-Without Nitrification	0.31 - 0.72	0.2 - 0.5	6 h	25 - 100	1.0 kg O_2/kg BOD_5	4 - 6	1000 - 3000
-With Nitrification	0.31- 0.72	0.05 - 0.25	6 h	50 - 200	1.0 kg O_2/kg BOD_5 + 4.6 kg O_2/kg TKN	>4 at 20°C >10 at 5°C	3000 - 5000
Extended Aeration (Provides Nitrification)	0.17 - 0.24	0.05 - 0.15	15[(10)] h	50 - 200	1.5 kg O_2/kg BOD_5 + 4.6 kg O_2/kg TKN	>15	3000 - 5000
High-Rate -Without Nitrification[(4)]	0.72 - 0.96	0.4 - 1.0	4 h	50 - 200	1.0 kg O_2/kg BOD_5	4 - 6	1000 - 3000
Contact Stabilization -Without Nitrification[(4)]	0.31 - 0.72[(5)]	0.2 - 0.5[(5)]	0.33[(6)] h 4[(7)] h	50 - 150	1.0 kg O_2/kg BOD_5	4 -10	1000 - 3000
Aerated Facultative Lagoons[(8.)]	0.031 - 0.048	-	4 -5 d	-	1.0 kg O_2/kg BOD_5	-	N/A

1. "F" is the mass loading to the aeration tank of BOD_5 per day and "M_v" is the mixed liquor volatile suspended solids mass under aeration.
2. Return sludge pumping should be variable over the full range given.
3. Including step aeration.
4. High-rate and contact stabilization not considered suitable for nitrification.
5. Based on contact and re-aeration tankage.
6. Based on Q Peak + 100% Q Avg. return sludge rate, (Contact).
7. Based on 100% Q Avg. return sludge rate, (Re-aeration).
8. Aerated facultative lagoons providing pretreatment prior to conventional lagoons (minimum total retention time of 30 days).
9. The designer should adjust these values to the necessary O_2 transfer rate of the chosen aeration equipment by applying factors for alpha, beta, DO and non-standard conditions such as altitude and temperature. U.S. customary units are lb O_2/ lb BOD_5 or TKN applied. The BOD_5 and TKN loadings refer to what is entering the aeration tank.
10. If nitrification is required year-round, a longer detention time may be required.
11. Deviations from the recommended design parameters may be considered if the designer can demonstrate through operating data or tests that the required treatment efficiency can still be consistently achieved.
12. Organic loading rates in US customary units are:
    a. 0.31-0.72 kg BOD_5/($m^3 \cdot d$) is 0.019-0.045 lb BOD_5/($ft^3 \cdot d$)
    b. 0.17-0.24 kg BOD_5/($m^3 \cdot d$) is 0.011-0.015 lb BOD_5/($ft^3 \cdot d$)
    c. 0.72-0.96 kg BOD_5/($m^3 \cdot d$) is 0.045-0.060 lb BOD_5/($ft^3 \cdot d$)
    d. 0.031-0.048 kg BOD_5/($m^3 \cdot d$) is 0.002-0.003 lb BOD_5/($ft^3 \cdot d$)
13. The above aeration system design parameters apply under a normal range of peak hourly flow to average daily flow ratio of 2 - 4:1.

12.2.4.4 Aeration Tank Capacities and Loadings

The size of the aeration tank should be determined by pilot plant studies, or calculations based on solids retention time, food-to-microorganism ratio and mixed liquor suspended solids levels. Other factors, such as size of treatment plant, diurnal load variations, degree of treatment required and data from similar full-scale STPs should be considered. In the case of the nitrification process, temperature, alkalinity, pH and DO concentration are important factors that the designer needs to consider.

Calculations should be carried out to justify the basis for design of aeration tank capacity. Calculations using values differing substantially from those in Table 12-1 should reference actual full-scale operational plants. Mixed liquor suspended solids (MLSS) levels greater than 5000 mg/L may be considered if pilot or other operational data shows that the aeration and clarification system are capable of supporting such high solids concentrations.

The aeration tanks are normally designed on average daily BOD_5 loading at the design average daily flow (*Section 8.5.11 - Design Basis for Various Plant Components*). When nitrification is required, the designer should evaluate the Total Kjeldahl Nitrogen (TKN) loading variations to prevent the effects of effluent ammonia bleed through during peak TKN loadings by establishing appropriate safety factors. A ratio of 7.14 mg/L alkalinity destroyed per mg/L of total ammonia nitrogen (TAN) should be used and a minimum residual alkalinity of 50 mg/L as calcium carbonate should be available. When process design calculations are not carried out, the aeration tank capacities and loadings for the processes shown in Table 12-1 should be used. The values apply to plants receiving daily load ratios of peak hourly BOD_5 to average daily BOD_5 ranging from about 2:1 to 4:1.

12.2.4.5 Arrangement of Aeration Tanks

Aeration basin depth affects the aeration efficiency and mixing capabilities of diffused aeration devices and mechanical aerators. The dimensions of each independent aeration tank or return sludge re-aeration tank should be such as to maintain effective mixing and utilization of air. Liquid depths should not be less than 3 m (10 ft) or more than 9 m (30 ft) except in special design cases such as horizontally mixed aeration tanks. An aeration basin depth of between 3.5 and 4.6 m (11.5 to 15.0 ft) is recommended. Complete mixed tanks have length-to-width (L/W) ratios of 1:1 to 3:1. Plug flow tanks have much larger L/W ratios of generally greater than 4:1, with baffling to simulate plug flow. Plug flow tanks provide the capability to perform step-feed, biological nutrient removal (with separate aerated and non-aerated zones) and improved nitrification kinetics. Arrangement of the aeration tanks in terms of the entire plant layout is discussed in more detail in *Section 8.1.3 - General Plant Layout* and *Section 8.1.4 - Provisions for Future Expansion*.

For very small tanks or tanks with special configuration, the shape of the tank, the location of the influent and sludge return and the installation of aeration equipment should provide for positive control to prevent short-circuiting through the tank.

Total aeration tank volume should be divided among two or more units, each capable of independent operation.

Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, stop plates, weirs, or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The effluent weir for a horizontally mixed aeration tank system should be easily adjustable by mechanical means and should be sized based on the design peak instantaneous flow plus the maximum return sludge flow. The hydraulic properties of the system should permit the design peak instantaneous flow to be carried through with any single aeration tank unit out of service. Overflow devices are preferred to avoid trapping foam and scum.

Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleansing velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits. Adequate provisions should be made to drain segments of channels, which are not being used due to alternate flow patterns.

All aeration tanks should have a freeboard of not less than 460 mm (18 in). However, if a mechanical surface aerator is used, the freeboard should be not less than 0.9 m (3 ft) to protect against windblown spray freezing on walkways.

12.2.4.6 Mixing Requirements

The aeration system which is selected should not only satisfy the oxygen requirements of the mixed liquor, but should also provide sufficient mixing to ensure that the mixed liquor remains in suspension. The designer should be aware that it is important to avoid both insufficient and excessive mixing. One exception to this is with aerated facultative lagoons where mixing is only provided to the extent necessary to ensure uniform DO levels in the upper layers of the aeration cell. The power transferred to the mixed liquor and diffused air flow rates to achieve uniform DO and MLSS concentrations are shown in Table 12-2.

12.2.4.7 Oxygenation Capacity

Consideration should be given to reducing power requirements of aeration systems by varying oxygenation capacity to match oxygen demands within the system. Such a system would utilize automatic DO probes in each aeration basin to measure dissolved oxygen levels. Consideration should be given to the operations and maintenance requirements to maintain these systems. An output signal could then be used to change the number of aerators in operation, aerator speed, immersion of surface aerator impellers, or air flow rate to submerged turbine and diffused aeration systems to maintain the required minimum DO levels.

Table 12-2– Aeration Mixing Requirements [1]

Aeration System	For Uniform DO Levels	For Uniform MLSS Levels
Mechanical	1.6 to 2.5 W/m^3 [(0.06 to 0.09 hp/($10^3 ft^3$)]	16 to 25 W/m^3 [0.61 to 0.95 hp/($10^3 ft^3$)]
Coarse Bubble Diffusers[2]	-	0.33 L/($m^3 \cdot s$) (0.02 cfm/ft^3)
Fine Bubble Diffusers[3]	-	0.61 L/($m^2 \cdot s$) (0.12 cfm/ft^2)

Notes:

1. Mixing requirements vary with tank or basin geometry, MLSS concentrations, placement of aeration devices and pumping efficiency of aerators. Wherever possible, the designer should refer to full-scale testing results for the particular aerator being considered.
2. L/($m^3 \cdot s$) refers to volume of air per second per volume of aeration tank.
3. L/($m^2 \cdot s$) refers to volume of air per second per horizontal cross-sectional area of aeration tank.

12.2.4.8 Aeration Equipment

Oxygen requirements depend on maximum diurnal organic loading, degree of treatment and level of total suspended solids concentration to be maintained in the aeration tank mixed liquor. Aeration equipment should be capable of maintaining a minimum of 2 mg/L of dissolved oxygen in the mixed liquor at all times and provide thorough mixing of the mixed liquor.

In the absence of experimentally determined values (recommended method), the design oxygen requirements for all activated sludge processes should be 1.0 kg O_2/kg design average daily BOD_5 (1.0 lb O_2/lb design average daily BOD_5) applied to the aeration tanks, with the exception of the extended aeration process, for which the value should be 1.5 kg O_2/kg design average daily BOD_5 to include endogenous respiration requirements.

Where nitrification is required or will occur, such as within the extended aeration process, the oxygen requirement for oxidizing ammonia should be added to the above requirement for BOD_5 removal and endogenous respiration needs. The nitrogenous oxygen demand (NOD) should be taken as 4.6 times the TKN content of the influent.

In addition, the oxygen demands due to recycle flows (e.g., anaerobic digester supernatant, heat treatment supernatant, dewatering centrate or filtrate, elutriates) need to be considered due to the high concentrations of BOD_5 and

TKN associated with such flows. Similarly, contaminant loads associated with any septage or landfill leachate additions to the STP for co-treatment need to be considered.

Careful consideration should be given to maximizing oxygen transfer per unit of power input. In some site-specific situations (e.g. wide variations in industrial loadings) the aeration system should be designed to match the diurnal organic load variation while economizing on power input.

The design requirements of an aeration system should accomplish the following:

- Maintain a minimum of 2.0 mg/L of DO in the mixed liquor at all times throughout the tank or basin;
- Maintain all biological solids in suspension (Table 12-2);
- Meet maximum oxygen demand and maintain process performance with the largest unit out of service; and
- Provide for varying the amount of oxygen transferred in proportion to the load demand on the STP.

12.2.4.9 Diffused Air Systems

Having determined the oxygen requirements as discussed earlier in this section, air requirements for a diffused air system should be calculated by incorporating such factors as:

- Tank depth;
- Alpha factor of sewage;
- Beta factor of sewage;
- Certified aeration device transfer efficiency;
- Minimum aeration tank DO concentration;
- Critical sewage temperature; and
- Altitude of plant.

In the absence of experimentally determined alpha and beta factors, sewage transfer efficiency should be assumed to be not greater than 50 percent of clean water efficiency (i.e., accounting for aeration parameters Alpha times Beta) for plants treating primarily (90 percent or greater) domestic sewage. Treatment plants where sewage contains higher percentages of industrial wastes should use a correspondingly lower percentage of clean water efficiency and should have calculations performed to justify such a percentage. The design transfer efficiency should be included in the specifications.

The designer should also consider the following:

- Additional air supply should be provided for aerated channels, air driven pumps and aerobic digesters above the air requirements for secondary treatment. Separate aeration supply for aerobic digester is preferred for control purposes;
- The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 45 °C (115 °F) or higher and the pressure may be less than normal. The specified capacity of the motor drive should also take into account that the intake air may be -30 °C (-20 °F) or less and may require over sizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor;
- The blowers should be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand with the single largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the plant. Aeration equipment should be easily adjustable in increments and should maintain solids in suspension within these limits;
- Diffuser systems should be capable of handling the air output from all blowers installed (including standby). The air diffusion piping and diffuser system should be capable of delivering normal air requirements with minimal friction losses;
- Air piping systems should be designed such that total head loss from blower outlet (or silencer outlet, where used) to the diffuser inlet does not exceed 3.4 kPa (0.5 psi) at average operating conditions;
- The spacing of diffusers should be in accordance with the oxygen uptake through the length of the channel or tank and should be designed to facilitate adjustment of the spacing without major revisions to air header piping;
- Individual assembly units of diffusers should be equipped with control valves, preferably with indicator markings, for throttling or complete shutoff. Diffusers in any single assembly should have uniform pressure loss; and
- Air filters should be provided in numbers, arrangements and capacities to furnish at all times an air supply sufficiently free from dust to prevent damage to blowers and clogging of the diffuser system.

12.2.4.10 Fine Bubble Diffuser Systems

With increased emphasis being placed on energy conservation in STP design, fine bubble diffuser systems are generally considered for new facilities and retrofits. Such systems have oxygenation efficiencies under process conditions of approximately 12 per cent with conventional tank depth (4.6 m) and greater efficiencies with increased depths.

Due to their increased oxygenation efficiencies, the air flows satisfying oxygen demand may not provide adequate mixing. The designer should ensure that proper mixing will occur. The fine bubble systems may under certain circumstances foul with slime. The slime formation is caused high F/M ratios, high soluble BOD_5 levels, low DO concentrations and also by low mixing levels.

The designer should also consider the following:

- Pilot testing should be carried out to determine if sliming will occur. This is particularly important where industrial waste contribution is significant or where soluble BOD_5 concentrations are expected to be high due to other causes;
- Manufacturer's recommended maximum and minimum air flow rates should be complied with;
- The ability to vary air flow rates should be possible to take full advantage of the efficiency of the diffusers and to minimize fouling; when satisfying peak oxygen demands, diffusers should be operating at close to their maximum air flow rating; air valves should be provided for each grid of the aeration system; automatic variation of air flow rate is desirable, but as a minimum a DO probe should be located in the area of the aeration tank near the raw sewage inlet and set to alarm if DO falls to 1 mg/L or less;
- To facilitate dome diffuser cleaning, equipment should be provided to allow for rapid tank draining, diffuser removal and diffuser cleaning; cleaning of ceramic domes may be carried out by hosing and scrubbing, steaming or acid cleaning, or combinations of these methods; a clean water source should be available to refill the tanks following cleaning;
- Air cleaning should be provided; replaceable air filters, using coarse pre-filters and fine final units may be the simplest and least expensive; electrostatic precipitators or bag houses may also be used; with retrofit plants, old piping may have to be replaced since the flaking of rust from cast iron lines may clog the diffusers from inside; equipment should be provided to remove liquid accumulations from inside the headers following power failure or repair shut downs;
- Spare parts should be provided including diffusers, gaskets, bolts and air supply piping;
- Since dome or disk diffusers are better vertical mixers than horizontal mixers, tanks should be built as deep as possible to minimize the horizontal travel required; oxygen transfer efficiency, however, may taper off at depths greater than 6.1 m due to oxygen depletion in the air bubbles; and

- Although diffusers work best under conditions of uniform loading, some degree of plug flow appears to be desirable for good sludge settleability; L/W ratios of approximately 8:1 are recommended; the aeration system should be divided into approximately 4 grids with the number of dome diffusers per grid being gradually reduced to match oxygen supply to demand; full-floor coverage should be provided in the first 3/4 of the aeration tank; in the last 1/4 of the aeration tank, the dome diffusers should be positioned along the centre line of the tank to induce a double spiral roll mixing effect; to avoid over-design of the oxygen supply, 50 percent blanks should be provided in at least the first half of the aeration system for possible addition of more diffusers in the future, if necessary; step feeding to at least mid-tank length should also be allowed for in design in case it is needed to reduce sliming problems.

Manufacturers' mixing power recommendations should be considered and compared to values in Table 12-2.

12.2.4.11 Mechanical Aeration Systems

The mechanism and drive unit should be designed for the expected conditions in the aeration tank in terms of the power performance. Certified testing should be provided to verify mechanical aerator performance. Design transfer efficiencies should be included in the specifications.

The design requirements of a mechanical aeration system should provide that motors, gear housing, bearings and grease fittings be easily accessible and protected from inundation and spray as necessary for proper functioning of the unit.

Where extended cold weather conditions occur, the aerator mechanism and associated structure should be protected from freezing due to splashing.

12.2.4.12 Return Sludge Equipment

The minimum permissible return sludge rate from the final sedimentation tank of the ASP is a function of the concentration of suspended solids in the mixed liquor entering it, the *sludge volume index* (SVI) of these solids and the length of time these solids are retained in the sedimentation tank. See *Chapter 13 – Secondary Sedimentation*.

Since undue retention of solids in the final sedimentation tanks may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of sludge return expressed as a percentage of the design average daily flow of sewage should generally be variable between the limits set forth in Table 12-1.

The RAS rate should be varied by means of variable speed motors, drives, or timers (small plants) to pump RAS at the recommended rates (Table 12-1). All designs should provide for flexibility in operation in various process modes, if feasible.

If motor driven return sludge pumps are used, the maximum return sludge capacity should be obtained with the largest pump out of service. A positive head should be provided on pump suctions. Pumps should have at least 80 mm (3 in) suction and discharge openings.

If air lifts are used for returning sludge from each sedimentation tank hopper, no standby unit may be needed if the design of the air lifts facilitate their rapid and easy cleaning and other suitable standby measures are provided (i.e., blower capacity). Air lifts should be at least 80 mm (3 in) in diameter. Due to air within the RAS pipe, direct flow measurement within the pipe is difficult.

Discharge piping should be at least 100 mm (4 in) in diameter and should be designed to maintain a velocity of not less than 0.6 m/s (2 ft/s) when return sludge facilities are operating at normal return sludge rates. Suitable devices for observing, sampling and controlling RAS flow from each sedimentation tank hopper should be provided.

Waste sludge control facilities should be provided so that the excess activated sludge may be wasted from the RAS lines or directly from the aeration tank. Wasting from the return sludge is more common and provides a more concentrated sludge; however wasting from the mixed liquor provides a simpler control. The waste pumps and pipelines should be sized based on the expected maximum sludge production rates and minimum sludge concentrations. Although continuous wasting is preferred, for non-continuous wasting the capacity of pumps and pipelines should be designed to handle the wasting rates expected.

Means for observing, measuring (i.e., flow rate and total), sampling and controlling waste activated sludge (WAS) flow should be provided. Waste sludge may be discharged to the primary sedimentation tank, sludge digestion tank, sludge thickening or dewatering processes, storage tank or any practical combination of these units.

12.2.5 Flow Monitoring

Flow monitoring devices should be installed in all activated sludge plants for raw sewage or primary effluent, return sludge, waste sludge and air to each aeration tank. For plants designed for design average daily sewage flows of 4000 m^3/d (1 mUSgd) or more, these devices should totalize and record, as well as indicate flows. Where the design provides for all return sludge to be mixed with the raw sewage (or primary effluent) at one location, then the influent flow rate to each aeration tank should be measured.

12.3 SEWAGE TREATMENT LAGOONS

12.3.1 General

This section provides design guidelines for sewage treatment lagoons (also referred to as waste stabilization ponds) capable of achieving equivalent to secondary treatment (annual average concentrations of 25 mg/L $CBOD_5$ and 30 mg/L TSS) or better. The sewage treatment lagoons are classified based on either the bioactivity type (facultative and/or aerated lagoons) or mode of operation (seasonal discharge or continuous discharge). Aerated lagoons are further classified based on their design and purpose as either aerated facultative lagoons, completely mixed aerated lagoons and post-aeration polishing cells. Most of the lagoons in Ontario are facultative lagoons with seasonal discharge. Combinations of various types of lagoons are also used based on site-specific needs.

Lagoons utilized for equalization, infiltration, evaporation and sludge storage are not discussed in this section.

Lagoon systems are often capable of providing secondary equivalent sewage treatment at a lower cost than mechanical STPs when land costs are considered. This is generally the case with rural small municipalities, where sufficient low cost land is available in the vicinity of the service area and where low permeability soils are available for lagoon cell construction.

Seasonal discharge lagoons have advantages over continuous discharge lagoons and mechanical sewage treatment plants where receiving streams experience insufficient flows during at least part of the year to provide adequate dilution for continuous effluent discharges or where downstream recreational water uses make summer effluent discharges undesirable.

It is generally accepted practice in Ontario to design sewage treatment lagoons based upon average daily sewage flow rates and BOD_5 loading and making no special allowance for net precipitation entering the cells.

12.3.1.1 Facultative Lagoons

At the feasibility or pre-design planning stage for facultative lagoons, the designer should consider the following:

- Possible nuisances such as odours, algae and vectors (e.g. mosquitoes). For the recommended land use surrounding lagoons refer to *Section 4.5 - Separation Distances between Sewage Works and Sensitive Land Use*;
- Whether the lagoon can be continuously discharged or needs to operate on a seasonal discharge basis;
- The minimum time and calendar dates for discharge of the lagoon cell contents;
- Industrial wastewater component and its effects on lagoon treatment. In some cases, it may be necessary to pretreat industrial wastewater;
- Whether phosphorus removal will be necessary and if required, to what level;
- Whether effluent ammonia and/or hydrogen sulphide concentrations will need to be reduced; and

- What discharge rates will be permitted from seasonal discharge lagoons and what provision may be required for controlling effluent discharge rates in proportion to the receiving stream flow rates.

Facultative lagoons that need to be discharged prior to or soon after the ice cover leaves the lagoon in the spring, may have hydrogen sulphide levels in the effluent high enough to cause fish toxicity in the receiving stream. In such cases, the designer should consider oxidation of hydrogen sulphide in the effluent in a post-aeration cell (*Section 12.3.1.7 - Post-Aeration Polishing Cell*).

Ammonia may be stripped from the lagoon contents during the summer months of high algal growth and pH. However, higher ammonia levels in spring especially under ice cover, cannot be effectively treated using a post-aeration cell. In such cases, the designer should consider the use of intermittent sand filters (*Section 12.3.6 - Intermittent Sand Filters*) or other ammonia removal technology.

12.3.1.2 Facultative Lagoons with Supplemental Aeration

In some cases supplemental aeration may be the most economical means of upgrading or expanding a facultative lagoon. The additional aeration will supplement the insufficient level of oxygen provided by photosynthetic activity and natural surface reaeration of the upper layer. The required level of supplemental aeration should be established under the site-specific conditions.

12.3.1.3 Seasonal Discharge

Facultative lagoons, when operated on a seasonal discharge basis with phosphorus removal by batch dosing with alum or iron salts, are able to achieve an effluent quality ($CBOD_5$ of 15 mg/L, TSS of 20 m/L and TP of 0.5 to 1.0 mg/L) comparable to conventional activated sludge plants with phosphorus removal. To achieve such quality, the lagoon cells should be ice free at the time of planned discharge so that batch dosing can be used for phosphorus removal. Continuous addition of alum to the raw sewage entering lagoon cells has not proven to be as effective as batch dosing; however, effluent TP of 1 mg/L can be achieved.

The ability to introduce raw sewage to all lagoon cells is desirable, but as a minimum there should be a capability to divide raw sewage flows among enough cells to reduce the design average BOD_5 loading to 22 kg/(ha·d) (20 pounds per acre per day) or less, at the mean operating depth in the primary cells.

The required hydraulic detention time should be determined using the volume between 0.6 m (2 ft) (recommended minimum operating depth) and the maximum operating depth of the entire lagoon system and the design average daily flow. The hydraulic detention time should not be less than:

- The hydraulic detention time as set by the area needed to meet the design BOD_5 loading;

- The largest number of consecutive days of a year when discharge is not allowed; and
- The number of days the lagoon is under ice cover.

12.3.1.4 Continuous Discharge

For continuous discharge facultative lagoons, the design average BOD_5 loading distribution should be similar to that of a seasonal discharge lagoon (*Section 12.3.1.3 - Seasonal Discharge)*.

Design variables such as lagoon depth, multiple units, detention time, supplemental aeration and additional treatment units should be considered with respect to effluent quality requirements for $CBOD_5$, TSS, *E. coli*, ammonia, hydrogen sulphide, DO and pH. The major factor in the design is the duration of the cold weather period where the lagoon contents are at temperatures of less than 5 °C (41 ºF).

During the summer/fall months, the presence of algae may considerably increase the effluent TSS and $CBOD_5$ concentrations. If the effluent quality criteria are provided as average monthly concentrations, the designer needs to consider the appropriate facilities for algae removal like microscreening (*Section 15.3 - Microscreening*) or intermittent sand filters (*Section 12.3.6 – Intermittent Sand Filters*).

12.3.1.5 Aerated Lagoons

Aerated lagoons can be classified into two categories depending on the degree of aeration achieving partial or complete mixing. These types of lagoons are generally used in conjunction with continuous discharge operations, but may also be a part of a seasonal discharge lagoon system.

12.3.1.6 Aerated Facultative Lagoons

Aerated facultative lagoons are designed and operated to ensure that enough oxygen is transferred to satisfy the applied BOD_5 loading and maintain an adequate dissolved oxygen level. The lagoon contents should be mixed sufficiently to maintain uniform DO levels throughout the aerobic layer. No attempt is made to supply enough mixing to maintain a uniform suspended solids concentration. Mixing is kept low enough to permit solids settling. Solids settling to the lagoon bottom will undergo anaerobic decomposition and the products of this decomposition are released and treated in the upper aerobic layers.

The most common application for the use of aerated facultative lagoons is as pretreatment of raw sewage prior to discharge into subsequent lagoons. With 4 to 5 days of retention time, typical effluent quality from an aerated facultative lagoon treating domestic sewage will generally be a $CBOD_5$ concentration of 60 mg/L, TSS of 100 mg/L and TP of 6 mg/L. With a total retention time of 30 days in the lagoon system, effluent quality (annual average concentrations) equivalent to that produced by conventional activated sludge treatment may be achieved.

Aerated facultative lagoon systems (i.e., aerated facultative lagoons plus subsequent lagoons) designed to treat domestic sewage should consist of two or more aerated cells. It is recommended that the first two cells should be of equal size. If more than two cells are proposed, any cell should not provide more than 50 percent of the total required volume.

12.3.1.7 Post-Aeration Polishing Cell

A post-aeration polishing cell may be considered in cases where the period of discharge from a facultative lagoon falls at the time when the lagoon may have significant ice cover or soon after ice melt, resulting in high hydrogen sulphide levels.

The following design criteria should be incorporated in the design of a post-aeration cell to permit biochemical oxidation of hydrogen sulphide (H_2S) and to minimize stripping of the gas:

- The cell should be 3 to 4 m deep (10 to13 ft); have a L/W ratio of 4:1 and provide at least 12 hours of retention time;
- The influent should be fed to the bottom of the cell and dispersed at a minimum of three locations in the first two-thirds of the cell; and
- The aeration system should consist of a fine bubble diffuser to ensure high oxygen transfer and minimize mixing. Oxygen should be supplied to provide both 1.2 kg O_2/kg $CBOD_5$ (1.2 lb O_2/lb $CBOD_5$) and 1.0 kg O_2/kg H_2S (1.0 lb O_2/lb H_2S).

12.3.1.8 Completely Mixed Aerated Lagoons

Completely-mixed aerated lagoons are another type of aerated lagoon system, wherein complete mixing is achieved within the lagoon cells. The aeration systems are designed in a similar way to those of activated sludge processes, except that earthen berm construction is used for the aeration basin.

The design cell depth should be 3 to 4.6 m (10 to 15 ft). This depth limitation may be adjusted depending on the aeration equipment, waste strength and climatic conditions.

The completely-mixed aerated lagoon should be followed by a sedimentation basin.

12.3.2 Aeration Equipment

Various types of aeration systems may be used, including bridge mounted mechanical surface aerators, floating mechanical surface aerators and diffused aeration using submerged diffusers or aeration tubing. Where extreme winter temperatures are experienced, submerged aeration systems are recommended. If mechanical surface aerators are used, they should be of the low speed bridge mounted type to avoid icing damage. Erosion protection will generally be required below mechanical aerators to prevent bottom scour. For a completely mixed cell, power requirement of the aeration equipment to maintain solids in suspension would control the power input to the system and meet the oxygen demand.

Aeration requirements will generally depend on the BOD_5 loading, degree of treatment required, temperature and the concentration of suspended solids to be maintained in the cell. The final sizing of the aeration equipment should be based on guaranteed performance by the equipment manufacturer with verification of mixing and oxygen dispersion capabilities of the proposed aerators.

The designer should ensure the operational reliability of the aeration system by providing the following:

- The blowers serving diffused air systems should be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand with the largest unit out of service;
- The air diffusion system for each aeration cell should be designed such that the largest section of diffusers can be isolated without losing more than 50 percent of the oxygen transfer capability within each cell;
- The floating or fixed mechanical aerators should be provided in sufficient numbers to enable the design oxygen demand of a particular cell to be satisfied with the largest capacity aerator in that cell out of service;
- At least two mechanical aerators should be installed in each primary cell for a mechanical aeration-based system; and
- A backup aerator should be provided. The backup aerator may be a complete uninstalled unit or a motor. In the latter case, a prop assembly (drive train) should be provided so that the installed aerator or parts can be easily removed and replaced.

Suitable protection from the elements should be provided for electrical controls, aerators and piping.

12.3.3 Design Considerations

The minimum number of cells should be two for small installations. Larger installations should have a minimum of three cells designed to facilitate both series and parallel operations.

The maximum sewage depth in facultative lagoons should be 1.8 m (6 ft) in primary cells. Greater cell depths can be used if preceded by supplemental aeration or mixing. The bottom 0.3 m (1 ft) of cell liquid depth (i.e., retained sediment) should be retained at the completion of lagoon drawdown..

The shape of all lagoon cells should be such that there are no narrow or elongated portions. Rectangular lagoons (length not exceeding three times the width) are considered most desirable; long dimension(s) should not align with prevailing wind direction. The maximum size of each lagoon cell should be 8 ha (20 acres), but 4 ha (10 acres) is preferred.

The hydraulic capacity for seasonal discharge lagoons should be sized to permit all cells to be discharged in the minimum time specified in the design but not less than a minimum rate of 150 mm (6 in) of lagoon water depth per day at the available head.

The hydraulic capacity for continuous discharge structures and piping should allow for a minimum of 250 percent of the design maximum day flow of the system or be at least equal to the expected future peak raw sewage pumping rate.

Effluent from each cell should be drawn from 0.3 m (1 ft) above the cell bottom. Outlets should lead to effluent chamber(s) which permit level regulation. All cells should be provided with an emergency overflow system to overflow when the liquid contents reach within 0.6 m (2 ft) of the top of the berms.

Cross connection piping between adjacent cells should be interconnected to permit flow between cells. Where cells are at or near the same elevation, the pipes should be valved. Where cell elevations differ significantly, the cross connection pipe should have a chamber with a weir to control flow from the higher cell. The valve or chamber should be provided with suitable locking devices and be located off the traveled portion of the top of the berm.

Short-circuiting in continuous discharge lagoons should be minimized, especially to avoid the need for effluent disinfection. Two or more cells should be provided and designed to allow series (as well as parallel) operation. Effluent piping should be as far removed as possible from inlet or cross connection piping. Wind induced currents should be considered when lagoon orientation is being selected. Installation of baffles or curtains can be considered to reduce short-circuiting.

12.3.4 Lagoon Construction

A soil consultant's report should be prepared to address the following:

- The suitability of the native soils for the proposed construction and the need for a liner;
- The maximum *groundwater* elevation and depth to bedrock;
- The soil strata which will be suitable for use (i.e., for forming the cell bottom and berm cores), soils to be removed and solids suitable for topdressing and estimates of their permeabilities; and
- The estimated initial clear water leakage rate which should be experienced from the cell structures.

When a lagoon is being considered for a site where leakage is expected and where there are nearby groundwater uses or *surface water* bodies which are likely to be adversely affected, the above factors need to be evaluated by a hydrogeologist, as part of a hydrogeological assessment in accordance with *ministry* Guideline B-7, *Incorporation of the Reasonable Use Concepts into Ground Water Management Activities.* A system of wells or lysimeters may be needed around the perimeter of the lagoon site to facilitate groundwater monitoring where needed.

Soil used in constructing the lagoon bottom (not including the seal) and dike cores should be relatively incompressible and tight and compacted at or up to 4 percent above the optimum water content as required based on the soils report.

Under certain soil circumstances, liners may be required in order to minimize excessive leakage. Where clay liners are used, precautions should be taken to avoid erosion and desiccation cracking prior to placing the system in operation.

Berms should have a minimum top width of 3.0 m (10 ft) to allow for access by liquid alum trucks and maintenance vehicles. Minimum freeboard above maximum lagoon operating level should be 0.9 m (3 ft). Berm slopes should not exceed 4:1 (horizontal:vertical) inside and 3:1 outside unless greater slopes are recommended by a soil consultant. Adequate provision should be made to divert stormwater runoff around the lagoons and protect lagoon embankments from erosion.

Influent lines may be located along the bottom of the lagoon with the top of the pipe just below the average upper elevation of the lagoon seal or liner. However, the full seal depth needs to be maintained below the bottom of the pipe and throughout the transition area from the bottom of the pipe to the lagoon bottom.In situations where pipes penetrate the lagoon seal, provisions to prevent seepage (such as anti-seep collars) need to be made. The lagoon site should be fenced and provided with a locked access gate of sufficient width to accommodate mowing equipment.

12.3.5 Control Structures and Interconnecting Piping

A manhole or vented cleanout wye should be installed prior to entrance of the influent line into the primary cell and should be located as close to the dike as topography permits. Its invert should be at least 150 mm (6 in) above the maximum operating level of the lagoon and provide sufficient hydraulic head without surcharging the manhole.

Flow distribution structures should be designed to effectively split hydraulic loads equally between the primary cells.

All primary cells should have individual influent lines which terminate approximately at the midpoint of the width and at approximately two-thirds of the length away from the outlet structure so as to minimize short-circuiting.

The influent line should discharge horizontally into a shallow, saucer-shaped depression. The end of the influent discharge line should rest on a suitable concrete apron large enough to prevent the terminal velocity at the end of the apron from causing soil erosion. A minimum size apron of 0.6 m (2 ft) square should be provided.

The designer should consider the use of multi-purpose control structures to facilitate normal operational functions such as drawdown and flow distribution, flow and depth measurement, sampling, pumps for recirculation, chemical additions and mixing and minimization of the number of construction sites within the dikes.

As a minimum, control structures should be:

- Accessible for maintenance and adjustment of controls;
- Adequately ventilated for safety and to minimize corrosion;
- Locked to discourage vandalism;
- Equipped with controls to permit sewage level and flow rate control and complete shutoff;
- Constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electrochemical reactions); and
- Located to minimize short-circuiting within the cell and avoid freezing and ice damage.

Recommended devices to regulate sewage level are valves, slide tubes or dual slide gates. Stop logs should not be used. Regulators should be designed so that they can be preset to prevent the lagoon surface elevation from dropping below the desired operational level.

12.3.6 Intermittent Sand Filters

Use of the intermittent sand filter (ISF) process is a viable method for polishing lagoon effluents. The process involves application of lagoon effluent on a periodic or intermittent basis onto the surface of a sand filter bed. As the lagoon effluent passes through the sand, suspended and soluble matter are removed through a combination of physical straining and biochemical transformations. A mature ISF is a complex ecosystem with the majority of the biochemical activity concentrated near the surface of the filter. This allows ammonia to be nitrified and a portion of the BOD_5 to be removed. A properly designed and operated ISF system provides a very high removal of BOD_5 and TSS and can produce a completely nitrified effluent with high dissolved oxygen. In Ontario, intermittent sand filters have been demonstrated to be functional only during warmer non-freezing periods.

Filter surface is typically flooded once or twice per day with lagoon effluent. The influent system should be capable of applying the tota1 dai1y hydraulic load in less than 6 hours to ensure maximum head development and maximum bed reaeration after drainage. The length of the filter run is controlled by the size of the sand, the hydraulic loading rate and the total suspended solids concentration in the lagoon effluent. Night time applications to the bed have been shown to significantly extend filter runs by inhibiting the growth of algae in effluent applied onto the filters and filter beds.

Typical ISF filter run lengths may range from 30 days with lagoon effluent TSS concentrations of greater than 50 mg/L to one year with low lagoon effluent solids. To allow flexibility for cleaning, all systems should have at least two filter beds (three are preferred), each designed to receive the total flow.

The depth of the sand in the filter bed should be 0.9 m (3 ft) initially to allow removal of the top 2 - 5 cm (1 - 2 in) layer during each cleaning cycle and replacement of that sand about once per year. The fi1ter should not be operated with less than 0.6 m (2 ft) of sand on the bed. The effective particle size of the sand should be 0.15 to 0.30 mm, with a uniformity coefficient of less than 5.

The sand layer should be underlain by a graded gravel layer to prevent intrusion of sand into the underdrain piping. A grave1 bed should be 0.3 m (1 ft) deep and contain 10 cm (4 in) of 6 mm (0.25 in) pea gravel on top, 10 cm (4 in) of 20 mm (0.8 in) gravel in the middle and 10 cm (4 in) of 30 mm (1.25 in) gravel at the bottom.

The underdrain piping should have maximum spacing of 1.5 m (5 ft) with minimum lateral pipe size of 15 cm (8 in) in diameter and connected to an outlet manifold. This manifold should be designed to allow complete drainage of the underdrain network so that air can circulate through the drain system into the filter bed. The base of the filter bed should be lined with clay or membrane liners.

A dosing basin with a siphon or electrically actuated valves and timer controls should be used to apply the lagoon effluent to the filter bed at a hydraulic loading rate of 500 L/(m^2·d) (12 USgpd/ft^2) at one or more equal dosings per day. The influent zone should be provided with a gravel splash pad using 50 mm (2 in) gravel.

When site topography permits, gravity flow or automatic dosing siphons should be used for application of lagoon effluent. The use of pumps is necessary when fi1ter beds are operated in series to lift the effluent to the second stage filter unit. The containing walls for the filter unit are earthen embankments, but concrete or other materials can be used for smaller systems where space is limited. Washing and reuse of the sand is feasible when local sources of low cost sand are not available.

In Ontario, the major application of intermittent sand filters has been for ammonia removal and polishing of lagoon effluents. The major limitations associated with intermittent sand filters are the large land area requirements for construction of the system, the need to periodically remove or replace the upper layers of sand on the bed and to either clean or dispose of the removed sand.

12.4 OTHER BIOLOGICAL SYSTEMS

12.4.1 General

Alternative biological systems include a wide range of suspended growth, fixed film and hybrid processes. Some of the processes described in this section are not common in Ontario or in Canada, however these may find common usage in the future. Some of these processes may include proprietary equipment or processes that will require coordination with the manufacturer or supplier of the technologies. Care should be taken to obtain sufficient pilot or full-scale process performance results consistent with the design conditions.

12.4.2 Sequencing Batch Reactors

The fill-and-draw mode of the activated sludge process commonly termed the Sequencing Batch Reactor (SBR) may be used in a similar fashion to the activated sludge process. Continuity and reliability of treatment equal to that of the continuous-flow-through modes of the activated sludge process should be provided. The SBR process uses control strategies that permit optimization of the system. Manufacturer input should be included in the design and sizing of these units. Provision for emergency maintenance (e.g. spare parts) to minimize downtime should be considered.

12.4.2.1 Design Considerations

The designer should consider the following:

- More than two tanks should be provided. Influent baffling using a baffle wall and adequate physical separation of the influent from the decanter is recommended for any basin which may operate with a continuous feed during the settle and decant phases. The baffling should direct the influent wastewater below the sludge blanket. Average horizontal velocities through each baffle wall opening should not exceed 0.3 m/s (1 ft/s);
- All SBR tanks should have a minimum freeboard of not less than 600 mm (24 in);
- The decantable volume and decanter capacity of the SBR system with the largest basin out of service should be sized to pass at least 75 percent of the design peak daily flow without changing cycle times. A decantable volume providing at least 4 hours retention time with the largest basin out of service based on 100 percent of the design peak daily flow is recommended;
- System reliability with any single tank unit out of service and the instantaneous delivery of flow should be evaluated in the design of decanter weirs and approach velocities. The treated effluent from each reactor should be free of scum and have a total suspended solids concentration of no greater than 30 mg/L at any time. Scum removal should be provided. An adequate zone of separation between the

sludge blanket and the decanter(s) should be maintained throughout the decant phase;

- Decanters should draw treated effluent from below the water surface and exclude scum or have a means to exclude scum and floatables;
- Protection against ice build-up on the decanter(s) and freezing of the discharge piping and decant valve(s) should be provided;
- Treatment facilities with fixed decanters, or any other system where the low-water depth cannot be adjusted quickly by the operator, should be designed to end the decant phase at a higher water level than other types;
- The water depth of any basin where simultaneous fill and decant may occur should be limited to not less than 3.7 m (12 ft) at the end of the decant phase. The minimum water depth can be reduced to 3 m (10 ft) for SBRs with non-continuous feed;
- Adequate means to accommodate basin dewatering should be provided. All sludge transfer and wasting pumps should be accessible for maintenance without dewatering the tank;
- The capability to transfer sludge between SBR tanks should be provided. If the decant pumps are used for sludge transfer, all solids in the decant piping need to be flushed and recycled back to the SBR;
- The blowers should be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand in the aerated portions of the fill/react and react phases of the cycle with the single largest unit out of service;
- Oxygen transfer rates from the aerators based on average water depth between the low-water level and the maximum water level should be considered to provide a DO residual of 2.0 mg/L during aeration. Credits for oxygen recovery through denitrification should only be considered for those systems designed to denitrify;
- Independent aeration mixing should be provided for all systems where biological phosphorus removal or denitrification is required. The mixing equipment should be sized to thoroughly mix the entire basin from a settled condition within 5 minutes without aeration;
- Downstream processes need to be sized to handle peak discharge rates that will occur during decant phase unless equalization is provided for decant flow;
- All 24-hour effluent quality composite samples for compliance reporting or monitoring plant operations should be flow-paced and include samples collected at the beginning and end of each decant phase; and

- Programmable logic controllers (PLC) should be provided. Multiple PLCs should be provided as necessary to ensure rapid process recovery or minimize the deterioration of effluent quality from the failure of a single controller. An uninterruptible power supply with electrical surge protection should be provided for each PLC to retain program memory (i.e., process control program, last-known set points and measured process/equipment status) through a power loss. A hard-wired backup for manual override should be provided in addition to automatic process control. Both automatic and manual controls should allow independent operation of each tank. In addition, a fail-safe control should be provided which cannot be adjusted by the operator allowing at least 20 minutes of settling between the react and decant phases.

12.4.2.2 Unit Sizing

Activated sludge process design considerations in *Section 12.2 - Activated Sludge Process*) should be reviewed. The aeration tank volumetric loading should not exceed 0.24 kg $BOD_5/(m^3{\cdot}d)$ (15 lb BOD_5/d/1000 ft^3). Design F/M ratios should be within the range of 0.05 to 0.1 d^{-1}. The reactor MLVSS and MLSS concentrations and aeration tank volumetric loading rate should be calculated at the low-water level.

12.4.3 Membrane Bioreactors

The Membrane Bioreactor (MBR) process consists of a suspended growth biological reactor (activated sludge system variation) integrated with a microfiltration or ultrafiltration membrane system. The key to the technology is the membrane separator which allows elevated levels of biomass in the reactor to degrade or remove the pollutants from the waste stream. These systems typically operate in the microfiltration or ultrafiltration range which results in removal of particles having a nominal size larger than 0.1 µm and 0.01 µm, respectively.

The benefits of these processes are consistent high effluent quality, reduced footprint and increased expansion capabilities within the same tankage and ease of operation. Tertiary quality effluent (Table 8-1) is the normal output of a membrane bioreactor. Virtually no solids are lost via the *permeate* stream and the unintentional wasting of solids is reduced. As a result, the sludge age can be very accurately determined. Nitrification for ammonia removal is easily achieved by optimizing reactor and sludge age to specific sewage characteristics and effluent requirements.

If required, denitrification can be achieved with MBR processes that operate at MLSS concentrations of 10,000 mg/L and higher. The mixed liquor rapidly becomes anoxic in the absence of a continuous stream of air. Furthermore, the high level of biomass ensures that at all times there are enough microorganisms in the anoxic zone to efficiently convert the nitrates into nitrogen gas.

12.4.3.1 Design Considerations

MBRs can be configured in a number of different ways. The two main configurations differ by those in which the membranes are submersed directly in the bioreactor and those which contain external membrane process tanks. When membrane modules are submersed into the bioreactor, they are in direct contact with the mixed liquor. A vacuum is created within hollow fiber or flat-plate membranes by the suction of a permeate pump. The treated effluent passes through the membrane, enters the hollow fibers or a permeate collection zone and is pumped out by the permeate pump. An air flow may be introduced into the bottom of the membrane module to create turbulence which scours and cleans the membrane surface to maintain a satisfactory permeate *flux*. The permeate (treated effluent) is then collected for reuse or discharge.

Externally-coupled membrane processes operate in a similar manner, however, the membranes are contained in a separate tank through which the mixed liquor from the bioreactor requiring filtration constantly flows. Recycled mixed liquor flow can provide cross-flow velocity required for membrane scouring and flux control. Air is often added for both treatment and membrane scouring purposes.

The main difference between the two MBR configurations lies in the membrane cleaning processes where membranes submersed within the aeration tanks should be removed or isolated for cleaning while externally-coupled membranes are cleaned by evacuating the membrane tanks and providing for equalization during the cleaning procedures within the main aeration tank.

Adequate pretreatment of raw sewage by either microscreening or fine screening may be required upstream of MBR processes in order to prevent operational difficulties (i.e., buildup of trash, fat, hair, lint, and other fibrous materials in the membrane modules and/or integrated aeration devices).

12.4.3.2 Unit Sizing

The biological components of the MBR process can be designed similar to the activated sludge process (*Section 12.2 - Activated Sludge Process*). Owing to the typical elevated MLSS concentrations, the MBR process has characteristic long solids retention times (SRTs).

The type and design of the membranes is dependent on the orientation of the membrane and the manufacturer. The membrane manufacturer should be consulted for particulars to their units and to provide verified design parameters for their units. Verified MBR design parameters should be based on pilot- and full-scale systems with an adequate period of operation. Special care should be given to peak flow operations and consideration for units being out-of-service for cleaning.

12.4.4 Biological Aerated Filters

The Biological Aerated Filter (BAF) process comprises submerged, granular media filters which treat sewage by biologically treating carbonaceous and nitrogenous matter using biomass growth fixed to the media and by physically capturing suspended solids within the media. No downstream secondary clarification is required.

BAFs are aerated to degrade carbonaceous biodegradable matter and convert ammonia-nitrogen to nitrates via nitrification. Non-aerated filters in the presence of supplemental organic matter can convert nitrates into nitrogen gas through denitrification.

BAFs are designed either as co-current backwash or countercurrent backwash systems. The co-current backwash design has a nozzle deck supporting a granular media that has a specific gravity (SG) greater than 1.0. Pretreated sewage is introduced under the nozzle deck and flows up through slightly expanded media bed and effluent leaves the filter from above the media. Process air is introduced just above the nozzle deck (the bed is not aerated for denitrification). During backwash, wash water and air scour are introduced below the nozzle deck and flow up through the bed. Wash water is pumped to the STP headworks or directly to solids handling.

The countercurrent backwash BAF operates under the same general principles, except that the granular media has a SG less than 1.0 (e.g. polystyrene bead media). Therefore, the media float and are retained from above by a screen. During backwash, wash water flows by gravity through the media. Process air is introduced below the media; therefore, scour air moves countercurrent to the wash water flow.

12.4.4.1 Design Considerations

The performance of BAFs in terms of allowable loading rates and effluent quality depends on influent sewage quality and temperature. In general, higher organic or suspended solids influent loadings result in higher effluent concentrations. Adequate water velocity is necessary to provide scouring of the media and biomass and for an even flow distribution across the media bed. Inadequate water velocity can result in premature bed plugging; this is especially the case for denitrification reactors in which the effects of air scouring are not present.

Factors that positively affect nitrification include:

- Warm sewage temperature;
- Adequate aeration and good air distribution; and
- Low BOD_5 and suspended solids loading.

Most manufacturers have estimated that solids production from the BAF process is comparable to that of a conventional activated sludge process. Effluent contaminant concentrations from a single BAF cell increases for approximately 30 minutes following a backwash event and therefore a minimum of four cells should be included in any design to dampen these spikes.

The nozzle deck features nozzles that prevent media loss and assist in evenly distributing flow across the bed. The reported media loss from the BAF system is less than 2 percent per year. The nozzle openings are slightly smaller than the media and require that influent be pretreated with a fine screen to prevent plugging. Headloss across the media bed can be more than 2 m (6.6 ft) prior to backwash. In existing installations, the filters are constructed above grade. The combination of the tall structure [6 m (20 ft)] and headloss across the bed requires influent pumping to the BAF in most situations. In addition, the co-current designs require pumping of wash water which is a significant, but intermittent, energy demand.

Process air is required in BAF cells that are removing carbonaceous organic matter ($CBOD_5$) and ammonia. The process aeration system consists of coarse- to medium-bubble diffusers on a stainless steel piping grid. The diffusers should be simple and reliable as possible because of the difficulty in accessing the aeration grid. Energy for process air can represent more than 80 percent of the energy demand of a BAF system.

12.4.4.2 Unit Sizing

The granular media bed for both BAF designs is typically 3 to 4 m (9 to 12 ft) deep with media size of 3 to 6 mm (0.012 to 0.024 in) in diameter. The specific surface area of media ranges from 500 to 2000 m^2/m^3 (150 to 610 ft^2/ft^3). Contact time in the media is typically 0.5 to 1.0 hour. The media bed is backwashed every 24 to 48 hours for 20 to 40 minutes using a wash water volume about three times the media volume. Backwash water from a single event is collected in a storage tank and returned to the head of the STP or directly to solids processing over a 1- to 2-hour period. Backwash water typically contains 400 to 1200 mg/L of suspended solids. The backwash water recycle flow can represent up to 20 percent of the influent sewage flow.

BAFs can operate in different process configurations, depending on the facilities, effluent goals and sewage characteristics. The process can follow primary sedimentation (with or without chemical addition) or an activated sludge system. Adequate pretreatment is required to ensure that the BAF media and nozzles do not become plugged. Enhanced primary treatment (with chemical addition and/or polymer) can assist the BAF process in providing combined organic removal and nitrification in a single-pass orientation. Following primary sedimentation, BAF cells can be operated for BOD_5 removal at loadings of between 2.5 to 5.0 kg $BOD_5/(m^3 \cdot d)$ [0.16 to 0.31 $lb/(ft^3 \cdot d)$] or, under lower loading rates (less than 1.5 kg $BOD_5/(m^3 \cdot d)$ [0.09 $lb/(ft^3 \cdot d)$]) for both carbonaceous BOD_5 and ammonia-nitrogen removal. A cell can operate in a nitrification mode following an activated sludge system or another BAF cell removing carbonaceous matter.

A denitrification biological filter process can follow either an activated sludge or BAF system that is nitrifying. Denitrification usually requires methanol addition and sewage flow velocities should be greater than 10 m/h (32.8 ft/h).

12.4.5 Trickling Filters

A trickling filter (TF) is a fixed film process that is suitable to biologically treat municipal sewage, although consideration needs to be given to the impact of temperature loss that occurs through the trickling filter during winter periods. Trickling filters should be preceded by effective primary sedimentation tanks equipped with scum and grease removal devices or other suitable pretreatment facilities. Solids separation is an important part of the TF process; accordingly, downstream secondary clarification is required. (*Chapter 13 – Secondary Sedimentation*)

Trickling filters should be designed to provide for reduction in carbonaceous and/or nitrogenous oxygen demand in accordance with established site-specific effluent quality requirements or to properly condition the sewage for subsequent treatment processes. Multi-stage TFs should be considered if required to meet more stringent effluent quality criteria.

12.4.5.1 Design Considerations

The influent sewage may be distributed over the filter by rotary distributors or other suitable devices, to ensure uniform distribution over the surface area.

For rotary distributors, reverse reaction nozzles, hydraulic brakes or motor-driven distributor arms should be provided to not exceed the maximum speed recommended by the manufacturer and to attain the desired media flushing rate.

For reaction-type distributors, a minimum head of 610 mm (24 in) is required between the low water level in the siphon chamber and centre of the arms. Similar allowance in design should be provided for added pumping head requirements where pumping to the reaction-type distributor is used. A minimum clearance of 300 mm (12 in) between media and distribution arms should be provided.

Influent sewage may be applied to TFs by siphons, pumps or by gravity discharge from preceding treatment units. Influent to the trickling filter should be continuous and therefore the piping system should be designed for recirculation as required to achieve the design efficiency. The recirculation rate should be variable and subject to plant operator control at the range of 0.5:1 up to 4:1 (ratio of recirculation rate versus design average daily flow). A minimum of two recirculation pumps should be provided.

Forced ventilation should be provided for covered trickling filters to ensure adequate oxygen for process requirements. The design of the ventilation facilities should provide for operator control of air flow depending on the outside seasonal temperature.

The piping system, including dosing equipment and distributor, should be designed to provide capacity for the design peak hourly flow, including recirculation.

The trickling filter media should be resistant to ultraviolet degradation, disintegration, erosion, aging, all common acids and alkalis, organic compounds, fungus and biological attack. Such media should be structurally capable of supporting a person's weight or a suitable access walkway should be provided to allow for distributor maintenance.

Trickling filter media should have a minimum depth of 1.8 m (6 ft) above the underdrains. Rock and/or slag filter media depths should not exceed 3 m (10 ft) and manufactured filter media depths should not exceed those recommended by the manufacturer. Ventilation needs to be provided and forced ventilation should be considered.

To ensure sufficient void clearances, media with specific surface areas of no more than 100 m^2/m^3 (30 ft^2/ft^3) are acceptable for filters employed for carbonaceous matter reduction and 150 m^2/m^3 (46 ft^2/ft^3) for second stage nitrification.

The underdrains should have a minimum slope of 1 percent. Effluent channels should be designed to produce a minimum velocity of 0.6 m/s (2 ft/s) at design average daily flow rates of application to the TF including recirculated flows.

The underdrain system, effluent channels and effluent pipe should be designed to permit free passage of air. The size of drains, channels and pipe should be such that not more than 50 percent of their cross-sectional area will be submerged under the design peak instantaneous flow, including proposed or possible future recirculated flows.

Provision should be made for flushing the underdrains unless high rate recirculation is utilized.

Appropriate valves, sluice gates, or other structures should be provided to enable flooding of trickling filters comprised of rock or slag media for filter fly control.

A freeboard of 1.2 m (4 ft) or more should be provided for tall manufactured filters to contain windblown spray. At least 1.8 m (6 ft) of headroom should be provided for maintenance of the distributor on covered filters.

All distribution devices, underdrains, channels and pipes should be installed so that they may be properly maintained, flushed or drained. Mercury seals should not be permitted for rotary distribution seals. Ease of seal replacement should be considered in the design to ensure continuity of operation.

Covers should be provided to maintain operation and treatment efficiencies at cold temperatures by avoiding excessive temperature drop through the TF.

12.4.5.2 Unit Sizing

Pilot testing is recommended to verify performance predictions based upon the various design equations, particularly when significant amounts of industrial wastes are present in the raw sewage.

Trickling filter design should consider peak organic load conditions including the oxygen demands due to recycle streams (e.g. anaerobic digester supernatant, heat treatment supernatant, dewatering filtrate) due to high concentrations of BOD_5 and TKN associated with such flows. The volume of media determined from either pilot plant studies or by the use of a design equations should be based upon the peak daily BOD_5 organic loading. Trickling filters are designed based on a wetting rate and organic loading. Wetting rates will vary from 40 to 60 $m^3/(m^2 \cdot d)$ (980 to 1,470 $USgpd/ft^2$). Total organic loading rates vary from 0.4 to 1.8 $kg/(m^3 \cdot d)$ [0.025 to 0.11 $lb/(ft^3 \cdot d)$]. These loadings are for organic removal only with media depths of greater than 3 m (10 ft). For combined organic removal and nitrification, lower loading rates are required. The performance is temperature dependent due to the impact on nitrification and cooling that occurs through the trickling filter tower.

An enhanced trickling filter process, called the trickling filter/solids contact (TF/SC) process has been used successfully in North America. This process includes a short duration aeration cell downstream of the trickling filter to change the characteristics of the effluent solids to a suspended growth-type effluent (i.e., improved quality owing to flocculent biomass) prior to secondary sedimentation and solids recycle. The contact time in the solids contact process is generally between 30 and 60 minutes. The process may also include a re-aeration of the return solids from the solids contact tank.

12.4.6 Rotating Biological Contactors

The rotating biological contactor (RBC) is a fixed-film process that may be used to provide secondary treatment and can also be operated in seasonal or continuous nitrification and denitrification modes.

12.4.6.1 Design Considerations

Considerations for the RBC process should include those relevant to biological processes and specifically the following:

- Pretreatment effectiveness including scum and grease removal;
- Maximum organic loading rate on active disc surface area; and
- Minimum detention time at design peak daily flow.

Sewage temperature affects RBC performance. Year-round operation requires that the RBC be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the sewage with the resulting loss of performance.

Enclosures should be constructed of a suitable corrosion resistant material. Windows or simple louvered mechanisms which can be opened in the summer and closed in the winter should be installed to provide adequate ventilation. To minimize condensation, the enclosure should be adequately insulated and/or heated. Forced ventilation should be supplied when the RBCs are contained within a building provided with interior access for personnel.

RBCs need to be preceded by effective primary sedimentation tanks equipped with scum and grease removal devices or pretreatment devices which provide for effective removal of grit, debris and excessive oil and grease prior to the RBC units.

Solids separation is an important part of the RBC process; accordingly, downstream secondary clarification is required. (*Chapter 13 – Secondary Sedimentation*)

The temperature of sewage entering any RBC should not drop below 5 °C (41 ºF) unless there is sufficient flexibility to decrease the hydraulic loading rate or the units have been increased in capacity to accommodate the lower treatment efficiencies and rates. Otherwise, insulation or additional heating should be provided to the plant.

Adequate flexibility in process operation should be provided by considering one or more of the following:

- Variable rotational speeds in first and second stages;
- Multiple treatment trains;
- Removable baffles between all stages;
- Positive influent flow control to each unit or flow train;
- Positively controlled alternate flow distribution systems;
- Positive airflow metering and control to each unit when supplemental air operation or air drive units are used;
- Use of air scouring, reverse rotation and chemical cleaning to control excess growth on media; and
- Recirculation of secondary clarifier effluent.

The arrangement of RBC shafts in a series of stages has been shown to significantly increase treatment efficiency, by making the process more plug-flow in nature. It is recommended that an RBC plant be constructed in at least four stages for each tank. Four stages may be provided on a single unit by providing baffles within the tank or by multiple tanks. Sewage flow to RBC units may be either perpendicular or parallel to the media shafts.

RBC units may be placed in either steel or concrete tanks with baffles when required and constructed of a variety of materials. The design of the tanks should include:

- Adequate structural support for the RBC and drive unit;
- Elimination of the "dead" areas;

- Satisfactory hydraulic transfer capacity between stages of units; and
- Considerations for operator safety.

The structure should be designed to withstand the increased loads which could result if the tank were to be suddenly dewatered with a full biological growth on the RBC units. The sudden loss of buoyancy resulting from unexpected tank dewatering could increase the bearing support loadings by as much as 40 percent.

Provisions for operator protection can be included in the tanks design by setting the top of the RBC tanks about 0.3 m (1.0 ft) above the surrounding floor and walkways, with handrails placed along the top of the tanks, to provide an effective barrier between the operator and exposed moving equipment. The high tank walls will also prevent loss or damage by any material accidentally dropped in the vicinity of the units and entering the tanks.

Except under special circumstances, high-density media should not be used in the first stage. Its use in subsequent stages should be based on appropriate loading criteria, structural limitations of the shaft and media configuration.

The peripheral velocity of a rotating shaft should be approximately 18 m/min (60 ft/min) for mechanically driven shaft and between 9 - 18 m/min (30 to 60 ft/min) for an air-driven shaft. Provision should also be made for rotational speed control and reversal.

A means for removing excess biofilm growth, such as air scouring or water stripping, chemical additives, rotational speed control/reversal should be provided. First-stage DO monitoring should be provided. The RBC should be able to maintain a measurable DO level in all stages.

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

Consideration should be given to providing recirculation of RBC effluent flow. This may be necessary during initial system start-up and when the inflow rate is reduced. If flow can be recycled through the sludge holding or treatment units and then to the RBC process, then the organic load from the sludge units can be imposed on the RBC process. This imposed load will help to maintain the biogrowth and, as a secondary benefit, help stabilize and reduce the sludge.

Load cells, especially in the first stage(s), can provide useful operating and shaft load data. Stop motion detectors, rpm indicators and clamp-on ammeters are also potentially useful monitoring instruments.

In all RBC designs, access to individual shafts for repair or possible removal should be considered. Bearings should also be accessible for easy removal and replacement if necessary. Where all units in a large installation are physically located in very close proximity, it may be necessary to utilize large off-the-road cranes for shaft removal. Consideration should be given to crane reach, crane size, and the impact of being able to drain RBC tanks and dry a unit prior to shaft removal when designing the RBC layout.

12.4.6.2 Unit Sizing

Unit sizing should be based on experience at similar full-scale installations or thoroughly documented pilot testing with the site-specific sewage. In determining design loading rates, expressed in units of volume per day per unit area of media covered by biological growth, the following parameters should be considered:

- Design flow rate and influent sewage strength;
- Percentage of BOD_5 to be removed;
- Percentage of influent BOD_5 which is soluble;
- Media arrangement, including number of stages and unit area in each stage;
- Rotational velocity of the media;
- Retention time within the tank containing the media; and
- Sewage temperature.

In addition to the above parameters, loading rates for nitrification will depend upon influent TKN, pH and allowable effluent ammonia nitrogen concentration.

Hydraulic loading to the RBCs should range between 75 to 155 L/(m^2·d) (1.8 to 3.8 USgpd/ft^2) of media surface area without nitrification and 30 to 80 L/(m^2·d) (0.73 to 2.0 USgpd/ft^2) with nitrification.

Organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg BOD_5/(m^2·d) [0.006 to 0.008 lb BOD_5/(ft^2·d)] or 0.012 to 0.02 kg BOD_5 soluble/(m^2·d) [0.0025 to 0.0041 lb BOD_5 soluble/(ft^2·d)]. Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of dissolved oxygen, nuisance organisms and deterioration of overall process performance. The structural capacity of the shaft, provisions for stripping biomass, consistently low influent levels of sulfur compounds to the RBC units, the media surface area required in the remaining stages and the ability to vary the operational mode of the facility may justify choosing a loading in the high end of the range when the operator can carefully monitor process operations.

For purposes of plant design, the optimum tank volume is measured as sewage volume held within a tank containing a shaft of media per unit of growth covered surface on the shaft, or liters per square metre (L/m^2). The optimum tank volume determined when treating municipal sewage of up to 300 mg/L BOD_5 is 0.042 L/m^2 (0.0010 US gal/ft^2), which takes into account sewage displaced by the media and attached biomass. The use of tank volumes in excess of 0.042 L/m^2 (0.0010 US gal/ft^2) does not yield corresponding increases in treatment capacity when treating sewage in this concentration range.

Based on a tank volume of 0.042 L/m^2 (0.0010 US gal/ft^2), the detention time in each RBC stage should range between 40 to 120 minutes without nitrification and 90 to 250 minutes with nitrification.

RBCs should operate at a submergence of approximately 40 percent based on total media surface area. To avoid possible shaft overstressing and inadequate media wetting, the liquid operating level should never drop below 35 percent submergence. Media submergence of up to 95 percent may be allowed if supplemental air is provided. A clearance of 10 to 25 cm (4 to 10 in) between the tank floor and the bottom of the rotating media should be provided so as to maintain sufficient bottom velocities to prevent solids deposition in the tank.

12.4.7 Integrated Fixed-film Systems

Integrated Fixed-film Systems (IFS), with or without activated sludge, are hybrid dual systems with suspended biomass and fixed-film growth processes. These systems can be configured either as single-pass processes with no recycles or with sludge recycle (i.e., RAS).

12.4.7.1 Design Considerations

The basic IFS concept involves a single-pass system and is to have continuously operating, non-cloggable fixed-film reactors with no need for backwashing or return sludge flows, low head-loss and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the sewage in the reactor or the attached-growth support media may be immobile within the reactor for some designs. In the case of free-moving carrier elements, movement is normally induced by coarse bubble aeration in the aeration zone, although fine bubble aeration systems have also been used, while mechanical mixing is utilized in an anoxic/anaerobic zone. For small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

Free-moving biofilm carrier elements are available in various materials, densities, geometries and sizes, and are generally made of polyethylene or polypropylene. For free-moving carrier elements, a screen is placed at the outlet of the reactor to keep the biofilm elements in the reactor. Agitation constantly moves the carrier elements over the surface of the screen and the scrubbing action prevents clogging. Upstream fine screening of raw sewage should also be considered for such designs.

Almost any size or shape of tank can be retrofitted with the IFS process. The amount of carrier elements in the reactor may be decided for each case based on the degree of treatment desired, BOD5, TKN and hydraulic loadings, temperature and oxygen transfer capability. The reactor volume is completely mixed and consequently there is no "dead" or unused space in the reactor.

Solids separation is an important part of the IFS process; accordingly, downstream secondary clarification is required. (*Chapter 13 – Secondary Sedimentation*)

Similar design considerations should be considered for the Integrated Fixed-Film Activated Sludge (IFAS) process, with the design based on the mixed liquor suspended solids and its mixing and aeration needs. The IFAS process includes a RAS stream to provide for activated sludge as well as fixed film biomass for biological treatment.

12.4.7.2 Unit Sizing

Organic loading rates for these reactors are typically in the order of 3.5 to 7.0 g BOD_5/m^2 of media surface area/d (0.0007 to 0.0014 $lb/ft^2/d$) for $CBOD_5$ removal and less than 3.5 g BOD_5/m^2 of media surface area/d for nitrification (0.0007 $lb/ft^2/d$) based on the protected surface area. For nitrification with the IFS process, the required media surface area will usually be dictated by TKN loading, TAN removal requirements and biological growth conditions in the reactor (e.g. temperature, pH, DO). The designer should consult vendors for design details.

12.4.8 Biological Nutrient Removal

There are many proprietary Biological Nutrient Removal (BNR) systems available and the designer should consult vendors for design details. Advantages and disadvantages of BNR system are:

Advantages

- No (or reduced) chemicals or dosage control needed;
- Reduced sludge production;
- Reduced metal concentrations in effluent and sludge;
- High phosphorus content in sludge which increases its fertilizer value;
- Improved sludge settleability and dewatering characteristics;
- Reduced oxygen requirements;
- Reduced process alkalinity requirements;
- Increased oxygen transfer efficiency in aeration basin; and
- Reduced effluent nitrogen concentration.

Disadvantages

- Effluent filtration may be necessary to achieve very low phosphorus concentrations (since effluent solids would have higher phosphorus content);

- Phosphorus release may occur in anaerobic digesters and get transferred back to the STP headworks via solids processing recycle streams;
- Foaming;
- Need for skilled operation; and
- Need for more monitoring.

12.4.8.1 Biological Phosphorus Removal

A number of process configurations for enhanced biological phosphorus removal (BPR) have been developed as alternatives to chemical phosphorus removal. Phosphorus is removed in BPR processes by incorporating phosphorus into cell mass in excess of metabolic requirements. The key to the biological phosphorus removal is the continuous exposure of the microorganisms to alternating anaerobic and aerobic conditions. Exposure to these alternating conditions provides favorable conditions for BPR organisms to proliferate in sufficient numbers. The sludge containing the excess phosphorus is either wasted or removed through a sidestream. The alternating exposure to anaerobic and aerobic conditions can be accomplished in the main biological treatment process, or "mainstream", or in the return sludge stream, or "sidestream". The BPR process may require chemical phosphorous removal (as a polishing and/or backup system) to achieve very low phosphorus levels. The BPR is often combined with nitrification and denitrification processes for control of nutrients.

Unit Sizing

Typical design criteria for biological phosphorus removal are provided in Table 12-3 and 12-4.

Table 12-3 – Design Criteria for Biological Phosphorus Removal

Design Parameter	Treatment Configuration	
	Mainstream	Sidestream
Food/Microorganism Ratio (kg BOD_5/(kg MLVSS·d))	0.2 - 0.7	0.1 - 0.5
Solids Retention Time (d)	2 - 25	10 - 30
MLSS (mg/L)	2000 - 4000	600 - 5000
Hydraulic Retention Time (hrs) - Anaerobic Zone - Aerobic Zone	1 - 3 0.5 - 1.5	8 - 12 4 - 10
Return Activated Sludge (% of Influent Flow Rate)	25 - 40	20 - 50
Stripper Underflow (% of Influent Flow Rate)	N/A	10 - 20

12.4.8.2 Biological Nitrogen Removal

The principal nitrogen conversion and removal processes are conversion of ammonia to nitrate by biological nitrification and removal of nitrogen by biological denitrification.

Nitrification

Biological nitrification consists of the conversion of ammonia to nitrite followed by the conversion of nitrite to nitrate. This process does not increase the removal of nitrogen from the sewage over that achieved by conventional biological treatment. Nitrification is used when treatment requirements call for removal of effluent ammonia or as a first step of a total nitrogen removal system. To achieve nitrification, all that is required is the maintenance of conditions suitable for the growth of nitrifying organisms. Nitrification can be achieved in either a single stage (combined with organics removal) or in a separate nitrification stage. In each case, suspended growth, attached growth or hybrid systems can be used.

Combined Nitrification/Denitrification

The removal of nitrogen by biological nitrification/denitrification is a two-step process. In the first step, ammonia is converted aerobically to nitrate (NO_3^-) (nitrification). In the second step, nitrates are converted to nitrogen gas (denitrification). The removal of nitrate by conversion to nitrogen gas can be accomplished biologically under anoxic conditions. The carbon requirements may be provided by internal sources, such as sewage and cell material, or by an external source.

12.4.8.3 Combined Biological Nitrogen and Phosphorus Removal

A number of biological processes have been developed for the combined removal of nitrogen and phosphorus. Many of these are proprietary and use a form of the activated sludge process but utilizing combinations of anaerobic, anoxic and aerobic zones or compartments to accomplish biological nitrogen and phosphorus removal. Process design criteria are outlined in Table 12-4.

Table 12-4 – Design Criteria for Combined Biological Nitrogen and Phosphorus Removal

Design Parameter	Value
Food-to-Microorganism Ratio (kg BOD_5/(kg MLVSS·d))	0.1 - 0.25
Solids Retention Time (d)	10 – 40
MLSS (mg/L)	2000 – 5000
Hydraulic Retention Time (hrs)	
- Anaerobic Zone	0.5 - 2.0
- Anoxic Zone (total)	0.5 – 10
- Aerobic Zone (total)	4 – 12
- Total	5 – 24
Return Activated Sludge (% of Influent Flow Rate)	25 - 100
Internal Recycle (if required) (% of Influent Flow Rate)	100 - 600

CHAPTER 13

SECONDARY SEDIMENTATION

CHAPTER 13

SECONDARY SEDIMENTATION

The suspended and attached growth processes require separation of biomass from the biological process effluent to produce the secondary quality effluent, and for return of the microorganisms to the bioreactor (i.e., in the case of the activated sludge process) and wasting of excess biomass (i.e., waste sludge from all process types). This can be achieved by secondary sedimentation. A summary of the design loadings for secondary sedimentation tanks is provided in Appendix V, which should be used in conjunction with the details in this chapter.

13.1 GENERAL

Secondary sedimentation tanks (also known as secondary or final clarifiers) should be designed for the larger surface area in accordance with either clarification or solids thickening requirements based on the appropriate surface overflow rates and solids loading rates, respectively.

The surface area requirements for clarification vary with the settling characteristics of the suspended solids in the bioreactor effluent. In the case of the activated sludge process (ASP), factors that can influence the settling characteristics are chemical addition to the mixed liquor for phosphorus removal, and nitrification

Circular, rectangular, or square clarifiers may be used. In selecting the clarifier shape, the designer should consider the following factors:

- Effective use of the site;
- Means of future expansion;
- Head loss through the system;
- Operational and maintenance issues; and
- Economics of tank construction, including inlet and outlet piping, and sludge and scum removal equipment.

13.1.1 Number of Units

Multiple units capable of independent operation are desirable and should be provided in all plants where design average daily flows exceed 380 m^3/d (0.1 mUSgd). Plants not having multiple clarifiers should include other provisions (e.g., access to portable units during maintenance) to ensure continuity of treatment and meeting required site-specific effluent quality criteria in terms of concentrations and loadings.

13.1.2 Flow Distribution

It is recommended that effective flow splitting devices and control appurtenances (e.g. gates, splitter boxes) be provided to permit even or adjustable proportioning of flow and solids loading to each clarifier, throughout the expected range of flows. Flow distribution devices should consider solids distribution and avoid creating solids imbalances when splitting the flow. One example of poor solids splitting may occur when a bend is located directly upstream of a flow split device, which causes solids to be pushed against the outside channel wall and towards one side of the flow split device. See *Section 3.13.2 - Flow Distribution.*

13.2 DESIGN CONSIDERATIONS

13.2.1 Dimensions

It is recommended that the minimum length of flow from inlet to outlet be 3.7 m (12 ft) unless special provisions are made to prevent short-circuiting. The vertical side-water depths (SWD) should be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs. Secondary sedimentation tanks should have a SWD between 3.6 and 4.6 m (12 and 15 ft).

Greater SWDs are recommended for secondary clarifiers in excess of 372 m^2 (4000 ft^2) surface area [equivalent to 21 m (70 ft) diameter for circular clarifiers] and for plants providing nitrification. Less than 3.7 m (12 ft) SWD may be considered for package plants having a design average daily flow of less than 95 m^3/d (25,000 USgpd), if justified based on successful operating experience. For circular clarifiers with sludge hoppers, a 1:12 bottom slope should be considered.

Rectangular tanks should satisfy the following geometrical ratios:

- Length: width (L/W) of 4:1, or greater; and
- Width: depth (W/D) of 1:1 to 2.25:1.

13.2.2 Surface Overflow Rates

13.2.2.1 Intermediate Sedimentation Tanks

Surface Overflow Rate (SOR) for intermediate sedimentation tanks, following the fixed-film reactor processes, should not exceed 60 $m^3/(m^2{\cdot}d)$ (1,470 USgpd/ft^2) based on design peak hourly flow (DPHF).

13.2.2.2 Final Sedimentation Tanks

Settling tests should be conducted wherever a pilot study of biological treatment is warranted by unusual sewage characteristics, treatment requirements, or where proposed hydraulic or solids loadings differ from the recommended guidelines in this section.

Activated Sludge Process

To perform properly while producing a concentrated return flow, activated sludge sedimentation tanks should be designed to meet thickening and solids separation requirements. Since the rate of recirculation of return activated sludge (RAS) from the final sedimentation tanks to the aeration or reaeration tanks is quite high in activated sludge processes, SOR and weir overflow rate should be adjusted for the various processes to minimize the problems with sludge loadings, density currents, inlet hydraulic turbulence and occasional poor sludge settleability. The size of the sedimentation tank should be based on the larger surface area determined for SOR, based on the DPHF, and peak daily solids loading rate (SLR).

Attached Growth Biological Reactors

SOR for sedimentation tanks following fixed-film processes such as trickling filters or rotating biological contactors should not exceed 50 $m^3/(m^2{\cdot}d)$ (1,200 $USgpd/ft^2$) based on the DPHF.

The design criteria for activated sludge and attached growth systems shown in the Table 13-1 should not be exceeded. For flat-bottom circular clarifiers and shallower clarifiers, reduced design SORs should be used.

13.2.3 Inlet Structures

Inlets and baffling should be designed to dissipate the inlet velocity, to distribute the flow uniformly and to prevent short-circuiting. It is recommended that channels be designed to maintain a velocity of at least 0.3 m/s (1 ft/s) at one-half of the design average daily flow. Corner pockets and dead ends should be eliminated and corner fillets or channeling should be used where necessary. It is recommended that provisions be made for elimination or removal of floating materials which may accumulate in inlet structures.

With circular basins having 100 percent sludge recirculation, the inlet well should not be less than 20 percent of the tank diameter and have a depth of 55 to 65 percent of the SWD. The maximum flow velocity to the centre inlet well should not exceed 1.0 m/s (3.3 ft/s) and the outflow velocity should not exceed 0.08 m/s (0.26 ft/s). Other inlet structures and feed systems (e.g., peripheral and spiral feed circular sedimentation tanks) exist and the designer should consider site-specific design considerations for appropriate selection.

With rectangular tanks, baffled inlet ports are generally used to achieve uniform flow distribution. Maximum inlet port velocities should be in the range of 0.08 to 0.16 m/s (0.26 to 0.52 ft/s).

Table 13-1 – Final Clarifier Recommended Loading Rates

Treatment Process	Surface Overflow Rate at Design Peak Hourly Flow[1]	Peak Solids Loading Rate[3]
	$m^3/(m^2 \cdot d)$ ($USgpd/ft^2$)	$kg/(m^2 \cdot d)$ ($lb/(day \cdot ft^2)$)
Conventional ASP (CAS), Step Aeration, Complete Mix, Contact Stabilization, Carbonaceous Stage of Separate-Stage Nitrification	50 (1200)[2]	240 (50)
Extended Aeration, Single-Stage Nitrification	40 (1000)	170 (35)
Two-Stage Nitrification	33 (800)	170 (35)
Activated Sludge with Chemical Addition to Mixed Liquor for Phosphorus Removal	37 (900)	As above, depending on the treatment process

Notes:

1 Based on influent flow only.

2 Plants designed to meet less than 15 mg/L suspended solids after secondary clarification should reduce the design surface overflow rate.

3 Clarifier peak solids loading rate should be computed based on the design peak daily flow plus the design maximum return sludge flow rate and the design MLSS under aeration.

13.2.4 Weirs

Outlet weirs should be provided with sufficient effective length and in locations such that the clarified effluent can be withdrawn from the tank without causing excessive localized upflow resulting in solids carryover. For conventional circular tanks, a peripheral weir is generally all that is required to provide a suitable weir loading rate. With rectangular tanks, multiple weirs (e.g. more than one perpendicular or finger weirs) will generally be required and these should be located away from the area of upturn of the density current. Wall baffling can be used to reduce the likelihood of upflow causing solids carryover. The use of interior baffles and peripheral baffles (Stamford baffles) should reduce short-circuiting and enhance flocculation.

Overflow weirs should be readily adjustable over the life of the structure to correct for differential settlement of the tank. Consideration should be given to cleaning, maintenance and replacement in the design of weir troughs.

Overflow weirs should be located to optimize actual hydraulic retention time and minimize short-circuiting. It is recommended that peripheral weirs be placed at least 0.3 m (1 ft) from the wall.

It is recommended that weir loadings not exceed those shown in Table 13-2.

If pumping is required, the pumps should be operated as nearly continuous as possible to avoid flow disturbances. This is most readily provided by the use of variable speed pumps. Weir loadings should be related to pump delivery rates to avoid short-circuiting.

Table 13-2 – Recommended Weir Loading Rates for Final Clarifier

Design Average Daily Flow	Loading Rate at Design Peak Hourly Flow $m^3/(m \cdot d)$ (USgpd/ft)
Equal to or less than 4000 m^3/d (1 mUSgd)	250 (20,000)
Greater than 4000 m^3/d (1 mUSgd)	375 (30,000)

Weir troughs should be designed to prevent submergence at DPHF and to maintain a velocity of at least 0.3 m/s (1 ft/s) at one-half design average daily flow.

13.2.5 Submerged Surface

The tops of troughs, beams and similar submerged construction elements should have a minimum slope of 1.4 vertical to 1 horizontal. The underside of such elements should have a slope of 1 to 1 to prevent the accumulation of scum and solids.

13.2.6 Units Out-of-Service

The ability for a unit to be dewatered or taken out-of-service should conform to the provisions outlined in *Section 8.5.16 - Component Backup Requirements.* It is recommended that secondary sedimentation tanks be designed to provide for distribution of the plant flow to the remaining tanks, when one tank is out-of-service and/or dewatered.

13.2.7 Freeboard

It is recommended that the walls of sedimentation tanks extend at least 150 mm (6 in) above the surrounding ground surface and be provided with not less than 600 mm (24 in) freeboard.

Additional freeboard or the use of wind screens is recommended where larger sedimentation tanks are subject to high velocity wind currents that would cause tank surface waves and inhibit effective scum removal.

13.2.8 Sludge Settleability

Sludge settleability determines the *capacity* of an activated sludge clarifier since it partly determines the sludge settling rate against which the effluent overflow rate acts. The common measure of settleability in the activated sludge process is the *sludge volume index* (SVI). Several models have been developed to relate SVI to sludge settling velocity. However, SVI is a poor procedure for mixed liquor suspended solids (MLSS) concentration of greater than 3,000 mg/L and the stirred SVI (sSVI) test should be used. Where possible, design of activated sludge clarifiers should be based on field measurement of sludge settling velocity using batch settling tests at varying initial suspended solid concentration. To control high SVI conditions, bioselectors (*Section 12.2.3 - Selectors)* should be considered for activated sludge plants.

13.2.9 Clarifier Enhancements (Inlet and Baffles)

Many types of clarifier enhancement are available to improve secondary clarification performance. These enhancements are used to reduce short-circuiting and enhance flocculation. An inlet flocculation zone can be used to dissipate energy in the influent to the tank, through design of the centre feed well for circular clarifiers or as target baffles for rectangular clarifiers.

Interior (ring) and peripheral (wall) baffles have also been shown to disrupt the density currents, thus avoiding short-circuiting and solids carryover, especially at elevated flows. An interior baffle supported off the sludge collection mechanism (circular tank) or attached to the walls (rectangular tank) can be used to dissipate inlet energy and disrupt the density current. A peripheral, wall or effluent baffle can be used to deflect the density current away from the effluent weir and avoid short-circuiting and solids carryover. Two types of effluent baffles are common, the McKinney baffle which is horizontal in orientation and the Stamford baffle which is oriented at a 45° angle.

13.3 SCUM AND SLUDGE REMOVAL

13.3.1 Scum Removal

Full-surface mechanical scum collection and removal facilities, including baffling, are recommended for all sedimentation tanks. Where freezing would cause equipment damage, provision should be made for removal or protection (e.g. by covering a clarifier) of scum collectors in the winter. The characteristics of scum, which may adversely affect pumping, piping, sludge handling and disposal, need to be recognized in design. Provisions should be made to remove scum from the liquid train of the sewage treatment plant and to direct it to the solids treatment process. Under certain conditions, such as in biological nutrient removal (BNR) facilities with separate scum/foam removal on the bioreactors, separate clarifier scum removal would be unnecessary.

13.3.2 Sludge Removal

Mechanical sludge collection and withdrawal facilities should be designed to ensure rapid removal of the settled solids or RAS, to avoid adverse effects on the sludge quality caused by anaerobic conditions. Sludge should not remain in the sedimentation basin for more than 30 minutes. Sludge scraper systems may consist of chain and flight type or traveling bridge type for rectangular sedimentation tanks. Rotary circular scraper mechanisms are used in circular tanks. Scraper mechanisms, both in rectangular and circular sedimentation tanks, need to avoid excess travel time that could lead to long sludge detention times in the sedimentation tank and possible anaerobic conditions and associated solids carryover and odour potential. The designer should consult the manufacturer for recommended rates of sludge collectors and flight speeds.

In large tanks, traveling bridge or rotary mechanisms should be equipped with suction sludge draw off pipes for rapid sludge removal. Suction withdrawal should be provided for activated sludge clarifiers over 18 m (60 ft) in diameter, especially for nitrifying ASPs. RAS rates should be adjustable for adequate control, including individual adjustments on suction collection pipes.

When the settled solids are scraped towards a hopper for removal in rectangular tanks, the hopper has normally been located at the inlet end of the tank. Other designs placing the hopper at tank mid-point, or at the effluent end, to take advantage of the density current, have also been used successfully.

13.3.2.1 Sludge Hopper

The minimum slope of the side walls should be 1.7 vertical to 1.0 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms should have a maximum dimension of 0.6 m (2 ft). Extra depth sludge hoppers for sludge thickening should not be used.

13.3.2.2 Cross Collectors

Cross-collectors serving one or more sedimentation tanks may be useful in place of multiple sludge hoppers.

13.3.2.3 Sludge Removal Pipeline

Each hopper should have an individually valved sludge withdrawal line at least 150 mm (6 in) in diameter. The static head available for withdrawal of sludge should be 760 mm (30 in) or greater, as necessary to maintain a 0.9 m/s (3 ft/s) velocity in the withdrawal pipe. Clearance between the end of the withdrawal line and the hopper walls should be sufficient to prevent bridging of the sludge. Adequate provisions should be made for rodding or back-flushing individual pipe runs and allowance for visual confirmation of return sludge flow. Piping should be provided to return sludge for further processing.

13.3.2.4 Sludge Removal Control

Separate secondary sedimentation tank sludge lines may drain to a common sludge well. Sludge wells equipped with telescoping valves or other appropriate equipment should be provided for viewing, sampling and controlling the rate of sludge withdrawal. A means of measuring the RAS flow rate should be provided.

Wherever possible, pipes discharging RAS or waste activated sludge (WAS) should be located to permit visual confirmation that sludge is being discharged. It is recommended that each sedimentation tank has its own sludge withdrawal lines to ensure adequate control of the sludge wasting rate for each tank.

13.4 PROTECTIVE AND SERVICE FACILITIES

All secondary sedimentation tanks need to be equipped to enhance safety for operators. It is recommended that such features include machinery covers, lifelines, stairways, walkways, handrails and slip resistant surfaces.

The design should provide for convenient and safe access to routine maintenance items such as gear boxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle areas and effluent channels.

Electrical equipment, fixtures and controls in enclosed settling basins and scum tanks, where hazardous concentrations of flammable gases or vapors may accumulate, need to meet the requirements of current *Electrical Safety* Code for Class 1, Zone 0 or Zone 1 (or Division 1 for existing installations as defined under the code), Group D locations (O.Reg. 164/99) made under the *Electricity Act 1998*.

It is recommended that the fixtures and controls be located so as to provide convenient and safe access for operation and maintenance. Adequate area lighting needs to be provided.

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CHAPTER 14
DISINFECTION

CHAPTER 14

DISINFECTION

This chapter describes the design and safety considerations for the most common types of disinfection systems and practices used in treating sewage effluents. Design guidelines for chlorination, dechlorination, ultraviolet (UV) irradiation and ozonation are presented in this chapter. A summary of the design criteria and factors for conventional disinfection is provided in Appendix V which should be used in conjunction with the details in this chapter.

14.1 GENERAL

14.1.1 Disinfection Requirements

As specified in *ministry* Procedure F-5-4, *Effluent Disinfection Requirements for Sewage Works Discharging to Surface Waters*, disinfection requirements apply to all municipal and private communal *sewage works* discharging to *surface waters* and require ministry approval under section 53 of the *Ontario Water Resources Act* (OWRA).

The ministry should be consulted to determine the disinfection requirements for effluent discharges from any sewage works (see *Section 8.2 - Establishment of Effluent Quality Requirements*). The ministry may allow for seasonal relaxation of or exemption from disinfection requirements on a site-specific basis.

14.1.2 Sewage Treatment Plants Effluents

Sewage treatment plant (STP) effluent, which includes all overflows from within the STP site, should not exceed a monthly geometric mean density of 200 *E. coli* organisms per 100 mL, unless the proponent (designer) of the new works can demonstrated on a site-specific basis that such a practice can be relaxed without undue adverse effects on downstream beneficial water uses.

14.1.3 Sewage Lagoons Effluents

Lagoons designed in accordance with the design guidelines contained in *Section 12.3 - Sewage Treatment Lagoons* at the recommended organic loading and hydraulic retention time, with two or more cells in series and operated to avoid short-circuiting do not generally require disinfection. The designer should note that exemption from disinfection requirements may not be allowed where lagoons discharge into receiving waters where water supplies or bathing beaches are directly affected by the lagoon effluent.

14.1.4 Combined Sewer Overflows

Combined sewer overflow (CSO) disinfection is required where the sewage discharge affects swimming and bathing beaches and other areas where there are public health concerns. The effluent quality requirement for disinfected CSO during wet weather is a monthly geometric mean density not exceeding 1000 *E. coli* organisms per 100 ml. This requirement may be modified by the regional staff of the ministry on a case-by-case basis due to site-specific conditions, as outlined in the ministry Procedure F-5-5, *Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems*.

14.1.5 Dechlorination Requirements

In cases where chlorination is used as the disinfection process, subsequent dechlorination of the sewage works effluents should be provided to ensure that effluent is non-toxic to aquatic organisms. Normal operation of de-chlorination equipment should provide for an excess of reagents to ensure the effective destruction of the total chlorine residual.

14.1.6 Continuous Disinfection

To ensure continuous disinfection, consideration needs to be given to operation of the disinfection process during power outages. This requires standby power capacity (*Section 8.7.1 - Emergency Power Supply Facilities*). In addition, regular maintenance and breakdowns need to be considered in the design to ensure continuous disinfection is maintained at all times.

14.2 CHLORINATION

14.2.1 General

Where chemical disinfectants (such as chlorine) are used, the designer should consider meeting microbiological effluent quality criteria, disinfectant residual after appropriate contact time and a limiting maximum disinfectant concentration in the effluent discharge. The disinfection process should be selected after due consideration of sewage characteristics, type of treatment process provided prior to disinfection, sewage flow rates, pH of sewage, effluent quality criteria, current applicable technologies, disinfectant demand, equipment, chemicals, power cost, and maintenance requirements.

Chlorination is the predominant effluent disinfection process in full-scale use in Ontario at the present time. If chlorination is used, it may be necessary to subsequently dechlorinate the effluent if the chlorine residual in the discharge would exceed the effluent limits or would impair the natural aquatic habitat of the receiving water body.

14.2.2 Forms of Chlorine

Chlorine is available for disinfection in gaseous, liquid (hypochlorite solution), and pellet (hypochlorite tablet) forms. The use of chlorine gas or liquid will depend mainly on the size of the STP, the chlorine dose required and the safety concerns of the user. Large quantities of chlorine, such as are contained in one-tonne cylinders and tank cars, can present a considerable hazard to plant personnel and to the surrounding area. Potential public exposure to chlorine and operational costs should be considered when making the final determination of disinfectant type.

Although small STPs in Ontario may use liquid sodium hypochlorite for chlorination, many larger plants use liquefied chlorine gas under pressure. Due to safety concerns with chlorine gas, the use of sodium hypochlorite is likely to increase. Designers should therefore consider both chemicals when evaluating chlorination alternatives.

In designing the chlorination system, chlorine application should be considered for points other than the chlorine contact tank, as follows:

- Influent sewer (for odour control);
- Return activated sludge (for bulking control);
- Overflow sewers (for emergency disinfection);
- Upstream of polishing filter (for control against biological growth in filter beds);
- Sludge thickeners (for odour control and maintaining sludge in fresh condition); and
- Effluent water recycled for re-use in plant operations such as pump seal, cooling, dilution and/or flushing water.

14.2.3 Dosage

The *capacity* of the disinfection systems needs to be adequate to produce a dose sufficient to meet the applicable microbiological effluent criteria specified by the ministry. Required disinfection capacity will vary, depending on the uses and points of application of the disinfection chemical. Chlorination system sizing and the number of units should be designed for the whole range of sewage flow rates and for the type of control to be used. System design considerations need to include the controlling sewage flow meter (i.e., its sensitivity and location), telemetering equipment and chlorination controls. For typical sewage, the dosage ranges shown in Table 14-1 may be used as a guide in sizing chlorination facilities.

Table 14-1 - Typical Chlorine Dosages for Varying Levels of Treatment

Level of Treatment	Chlorine Dosage[1] (mg/L)
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. Based on design average daily flow.

14.2.4 Design Considerations

A total chlorine residual of 0.5 mg/L after 30 minutes of contact time at the design average daily flow is generally needed for disinfection of secondary treatment effluent to ensure that the monthly geometric mean density of *E. coli* does not exceed 200 organisms per 100 millilitres in the effluent discharge from the STP.

The designer of a chlorination system should ensure that minimum contact times of 30 minutes at design average daily flow and not less than 15 minutes at design peak hourly flow or maximum rate of pumping are provided after thorough mixing. For evaluation of existing chlorine contact tanks, field tracer studies are recommended. For effluent streams with higher microbial counts than typical secondary effluents, a higher chlorine residual and/or contact time may be required.

The disinfectant should be mechanically mixed as rapidly as possible, with an approximately complete-mix condition achieved within 3 seconds. This may be accomplished by either the use of turbulent flow regime or a mechanical flash mixer.

Actual contact time can be significantly different from calculated hydraulic retention time (HRT). The contact time should be provided in a pipeline or a tank where plug flow conditions are closely approached. Approximately plug flow regime can be reached in flow channels with length-to-width (L/W) ratios of greater than 40:1. L/W ratios of 10:1 produce contact times of approximately 70 percent of theoretical retention times. The height-to-width ratio of the wetted cross section of the channel should not exceed 2:1. In rectangular tanks, longitudinal baffling to produce long, narrow flow channels with a serpentine flow pattern and with guide vanes at changes in direction should be used to produce an efficient contact basin.

The tank should be designed to facilitate maintenance and cleaning without reducing effectiveness of disinfection. This is necessary since some sedimentation occurs in chlorine contact tanks. Duplicate tanks, mechanical scrapers, or portable deck-level vacuum cleaning equipment should be provided. Consideration should be given to providing skimming devices on all contact tanks. Covered tanks are not recommended.

In calculating the contact time, a contact time in the outfall sewer may be taken into consideration. If the outfall sewer is able to provide the full contact time, provision should at least be made for adequate mixing of the chlorine and sewage effluent prior to entering the outfall pipe and facilities provided so that chlorinated effluent samples can be obtained. These requirements can normally be satisfied by constructing a short-retention mixing chamber immediately upstream of the outfall sewer.

14.2.5 Containers

14.2.5.1 Cylinders

Sixty-eight (68) kg (150 lb) cylinders are typically used where chlorine gas consumption is less than 68 kg/d (150 lb/d). Cylinders should be stored in an upright position with adequate support brackets and chains at two-thirds of the cylinder height for each cylinder.

14.2.5.2 Ton Containers

The use of 907 kg (1 US ton) containers should be considered where the average daily chlorine consumption is greater than 70 kg/d (154 lb/d).

14.2.5.3 Liquid Hypochlorite Solution

Storage containers for hypochlorite solution should be of sturdy, non-metallic lined construction. Storage containers should be provided with secure tank tops and pressure relief and overflow piping. The overflow should be provided with a water seal or other device to prevent tanks venting to the indoors. Storage tanks should be either located or vented to the outside. Provision should be made for adequate protection from sunlight and extreme temperatures. Tanks should be located where leakage will not cause corrosion or damage to other equipment. A means of secondary containment needs to be provided to contain spills and facilitate cleanup. Due to deterioration of hypochlorite solution over time, it is recommended that containers not be sized to hold more than a one-month supply. At larger facilities and locations where delivery is not a problem, it may be desirable to limit on-site storage to one week.

14.2.5.4 Dry Hypochlorite

Dry hypochlorite should be kept in tightly closed containers and stored in a cool, dry location. Some means of dust control should be considered, depending on the size of the facility and the quantity of hypochlorite used.

14.2.6 Equipment

All chlorination facilities should be designed according to recommendations of the Chlorine Institute (http://www.chlorineinstitute.org). Scales for weighing cylinders and containers need to be provided at all STPs using chlorine gas. At large plants, scales of the indicating and recording type are recommended. At least a platform scale needs to be provided; scales should be of corrosion-resistant material. Scales or level sensing equipment are required for liquid chlorine systems.

Where manifolding of several cylinders or one-tonne containers will be required to evaporate sufficient chlorine, consideration should be given to the installation of evaporators to produce the quantity of gas required.

Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for chlorine service, with a minimum number of joints. Piping should be well supported and protected against temperature extremes.

Due to the corrosiveness of wet chlorine, all lines designated to handle dry chlorine need to be protected from the entrance of water or air containing water. Even minute traces of water added to chlorine results in a corrosive attack. Low pressure lines made of hard rubber, saran-lined, rubber-lined, polyethylene, polyvinyl chloride (PVC), or other approved materials are satisfactory for wet chlorine or aqueous solutions of chlorine.

It is recommended that the chlorine system piping be colour coded and labeled to distinguish it from other plant piping. Where sulphur dioxide is used for dechlorination, the piping and fittings for chlorine and sulphur dioxide systems should be designed, colour coded and labeled so that interconnection between the two systems cannot occur (*Section 8.7.3 - Plant Piping*).

Standby equipment of sufficient capacity should be available to replace the largest unit during shutdowns, including liquid feed pumps. Spare parts should be available for all disinfection equipment to replace parts which are subject to wear and breakage. Vacuum-operated automatic switchover devices that change from an empty to a full supply of chlorine should be provided (except on rail tank cars, where operator attendance is required). Also, an automatic switchover should be provided to activate the standby chemical pump upon a failure of the duty pump.

Chlorine injector systems require large volumes of water which typically amount to approximately 330 L of water per kilogram of chlorine used (40 US gal/lb). Higher water requirements can be experienced depending upon the back pressure at the point of injection. To minimize operating costs, filtered effluent water from the STP should be used in the injection systems whenever possible, with municipal water, or water from an on-site system, providing the necessary standby supply. Where a booster pump is required, duplicate equipment should be provided, and, when necessary, standby power as well. Protection of a potable water supply should be ensured by an air gap (*Section 8.7.2 - Water Supply*).

A bottle of 56 percent ammonium hydroxide solution needs to be available for detecting chlorine leaks. Where 907 kg (1 US ton) containers or tank cars are used, a leak repair kit approved by the Chlorine Institute needs to be provided. Consideration should be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking 907 kg (1 US ton) containers where such containers are in use. Scrubbers can be both dry and wet type. The designer should refer to Chlorine Institute guidelines for suitable equipment to neutralize chlorine. Other cylinders and tanker cars should also be provided with repair kits. Automatic gas detection and related alarm equipment need to be provided. Storage area for 907 kg (1 US ton) cylinders or containers should be provided with an overhead monorail hoist and motorized trolley of at least 1800 kg (2 US ton) capacity. The monorail should be of sufficient length to allow removal of the container without being rolled along the ground.

14.2.7 Housing

If gas chlorination equipment or chlorine cylinders are to be located in a building used for other purposes, a gastight room separating this equipment from any other portion of the building needs to be provided. Floor drains from the chlorine room should not be connected to floor drains from other rooms. Doors to this room need to open only to the outside of the building, and be equipped with panic hardware. Rooms need to be at ground level and should permit easy access to all equipment.

Storage areas for 907 kg (1 US ton) cylinders should be separated from the dosing area. In addition, the storage area should have designated areas for "full" and "empty" cylinders. Chlorination equipment should be situated as close to the application point as reasonably possible. Storage facilities should be designed to accommodate deliveries avoiding the need to have access to the storage area during deliveries.

A clear glass, gastight, window needs to be installed in an exterior door or interior wall of the chlorinator room to permit the units to be viewed without entering the room. In large facilities a glassed entry room provided with positive air pressure for viewing the storage area and providing storage for safety equipment should be considered.

It is recommended that rooms containing disinfection equipment be provided with a means of heating so that a temperature of at least 16 °C (60 °F) can be maintained. The room should be protected from excess heat. Cylinders should be kept at essentially room temperature. If liquid hypochlorite solution is used, the containers may be located in an unheated area. Rooms containing chlorination equipment are to be provided with ambient chlorine gas detectors. The gas detector should be interlocked with the fan and audible or visible alarms.

With chlorination systems, forced, mechanical ventilation needs to be installed that will provide 30 air changes per hour under emergency conditions and three air changes per hour under normal conditions in the room. The entrance to the air exhaust duct from the room should be near the floor. The point of discharge needs to be so located as not to contaminate the air inlet to any buildings or present a hazard at the access to the chlorinator room or other inhabited areas. Air inlets need to be so located as to provide cross ventilation with air and at such temperature that will not adversely affect the chlorination equipment. It is recommended that the outside air inlet be at least 1 m (3 ft) above grade. The vent hose from the chlorinator needs to discharge to the outside atmosphere above grade as should vents from feeders and storage areas. Where public exposure may be extensive, scrubbers may be required on the ventilation discharge. All chlorination facility ventilation systems should be designed according to recommendations of the Chlorine Institute (http://www.chlorineinstitute.org).

Switches for fans and lights should be located outside of the room at the entrance. A labeled signal light indicating fan operation should be provided at each entrance, if the fan can be controlled from more than one point. Consideration should be given to providing control such that the doors to the facility have an electrical interlock that automatically turns on the lights and exhaust fan in the room before entry and when the doors are opened. The ventilation fan could also be interlocked with the ambient chlorine gas detector to lock out operation of the fan in case of chlorine leak, to reduce dispersion of chlorine to the atmosphere.

Respiratory air-pack protection equipment, meeting the requirements of the Canadian Standards Association (CSA-Z94.4) governs and needs to be available where chlorine gas is handled, and needs to be stored at a convenient location, but not inside any room where chlorine is used or stored. Instructions for using the equipment need to be posted. It is recommended that the units use compressed air, have at least a 30-minute capacity and be compatible with the units used by the fire department responsible for the plant.

14.2.8 Sampling and Control

Facilities should be included for sampling disinfected effluent after the contact chamber or effluent outfall (if used for part of the required contact time). In large installations, or where stream conditions warrant, provisions should be made for continuous monitoring of effluent chlorine residual with *continuous monitoring equipment*.

The installation of demonstrated effective facilities for automatic chlorine residual analysis, recording, and proportioning systems should be considered at all large installations.

An automated dosage control system should be used for all sewage treatment facilities. The controls should adjust the chlorine dosage rate within an appropriate lag time to accommodate fluctuations in effluent chlorine demand and chlorine residual due to changes in flow and STP effluent characteristics. This may be accomplished using either closed-loop or feedback control methods. Alarms and monitoring equipment are required to promptly alert the operator in the event of any malfunction, hazardous situation, or inadequately disinfected effluent associated with the chlorine supply, including metering equipment, leaks or other problems.

14.2.9 Chlorine Safety Requirements

All chlorination facilities should be designed according to the recommendations of the Chlorine Institute. In addition, the design of the use, storage and handling of any hazardous materials should be in accordance with the *Occupational Health and Safety Act (*OHSA*)*, *Building Code*, (O. Reg. 350/06) made under the *Building Code Act, 1992* and *Fire Code* (O. Reg. 388/97) made under the *Fire Protection and Prevention Act, 1997*.

Chemical buildings or storage areas should be provided with adequate warning signs, conspicuously displayed where identifiable hazards exist, a storage area for *Material Safety Data Sheets (MSDS)* as set out under the federal *Hazardous Products Act* and associated *Controlled Products Regulations*. All storage containers should be conspicuously labelled with a Workplace Hazardous Materials Information System (WHMIS) label that includes: the product name, the supplier name, hazard symbol(s), risk, precautionary measures and first aid measures.

14.3 DECHLORINATION

14.3.1 Types of Dechlorination Agents

Dechlorination of sewage effluent may be required to meet site-specific effluent quality criteria set by the ministry to eliminate chlorine residual toxicity. The most common dechlorinating chemicals are sulphur compounds, particularly sulphur dioxide gas or aqueous solutions of sulphite or bisulphite. Pellet dechlorination systems are also available for small STPs.

The type of dechlorination system should be selected with consideration to the effluent quality criteria, type of chemical storage required, amount of chemical needed, ease of operation, compatibility with existing equipment, and safety.

14.3.2 Dosage

The dosage of dechlorination chemical should depend on the residual chlorine in the effluent, the final residual chlorine limit, and the particular form of the dechlorinating agent used. The most common dechlorinating agent is sulphite. Table 14-2 shows the forms of the compounds that are commonly used and dosage required to neutralize 1 mg/L of residual chlorine.

Table 14-2 - Forms of Dechlorination Chemicals

Dechlorination Chemical	Available as:	Theoretical Dosage Required to Neutralize 1 mg/L Cl_2 (mg/L)
Sodium sulphite	Tablets or powder	1.78
Sulphur dioxide	Liquefied gas under pressure	0.90
Sodium meta bisulphite	Solution or powder	1.34
Sodium bisulphite	Solution or powder	1.46
Sodium thiosulphate	Solution or powder	0.56

Theoretical values should be used for initial approximations, to size feed equipment with the consideration that under good mixing conditions, 10 percent excess dechlorinating chemical is required above theoretical values. Excess sulphur dioxide may consume oxygen at a maximum of 1.0 mg dissolved oxygen for every 4 mg sulphur dioxide (SO_2). Excess sulphur dioxide can impact the dissolved oxygen (DO) levels in a plant effluent, requiring better control or effluent re-aeration.

The liquid solutions are available in various strengths. These solutions may need to be further diluted to provide the proper dose of sulphite while matching the dosing pump range.

14.3.3 Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in 190 L (50 US gal) drums or dry compounds. Dilution tanks and mixing tanks will be necessary when using dry compounds and may be necessary when using liquid compounds to deliver the proper dosage. Solution containers should be covered to prevent evaporation and offensive odours.

14.3.4 Feed Equipment, Mixing, and Contact Requirements

In general, the same type of feeding equipment used for chlorine gas may be used with minor modifications for sulphur dioxide gas (*Section 14.2.6 - Equipment*). The manufacturer should be contacted for specific equipment recommendations. Automatic gas detection and related alarm equipment need to be provided. No equipment should be alternately used for the two gases. The common type of dechlorination feed equipment utilizing sulphur compounds include vacuum solution feed of sulphur dioxide gas and a positive displacement pump for aqueous solutions of sulphite or bisulphite.

Selection of the type of equipment for feeding sulphur compounds should give consideration to operator safety and overall public safety relative to the STP's proximity to populated areas and the security of gas cylinder storage. The selection and design of sulphur dioxide (SO_2) feeding equipment needs to take into account that the gas re-liquefies quite easily. Special precautions should be taken when using 907 kg (1 US ton) containers to prevent re-liquefaction.

Where necessary to meet the operating ranges, multiple units should be provided for adequate peak capacity and to provide a sufficiently low feed rate on turn down to avoid depletion of the dissolved oxygen concentrations in the receiving waters.

The dechlorination reaction with residual chlorine occurs within 15 to 20 seconds. The dechlorination chemical should be introduced at a point in the process where the hydraulic turbulence is adequate to ensure thorough and complete mixing. If no such point exists, mechanical mixing needs to be provided. The high solubility of SO_2 prevents it from escaping during turbulence.

A minimum of 30 seconds for mixing and contact time needs to be provided at the design peak hourly flow or maximum rate of pumping before a sampling point. Consideration should be given to a means of reaeration to ensure maintenance of an acceptable DO concentration in the sewage effluent following sulphonation.

14.3.5 Housing Requirements

The requirements for housing SO_2 gas equipment need to follow the same guidelines as used for chlorine gas handling (*Section 14.2.7 - Housing*).

When using liquid solutions for dechlorination, the solutions should be stored in a room that meets all safety and handling requirements. The mixing, storage, and solution delivery areas should be designed to contain or route solution spillage or leakage away from traffic areas to an appropriate containment unit.

The respiratory air-pack protection equipment is the same as for chlorine gas handling. Leak repair kits of the type used for chlorine gas that are equipped with gasket material suitable for service with sulphur dioxide gas may be used (*Section 14.2.7 - Housing*). (Refer to The Compressed Gas Association publication *Sulphur Dioxide*, CGA G-3-1995.)

14.3.6 Sampling and Control

Facilities need to be included for sampling the dechlorinated effluent for measurement of residual chlorine. Provisions should be made to monitor for DO concentration after sulphonation.

Manual or automatic control of sulphonator feed rates may be based on flow, chlorine residual, or sulphite or sulphate residuals measurements. Selection of on-line residual monitoring is dependent on effluent quality, size of the STP and operator skill level.

14.4 ULTRAVIOLET IRRADIATION

14.4.1 General

Design standards, operating data, and experience for the ultraviolet (UV) irradiation process are developed, but still need careful consideration, including evaluating the type of system and lamps to be used. Therefore, expected performance of the UV disinfection units need to be based upon experience at similar full-scale installations or thoroughly documented prototype testing with the particular sewage or independent third-party bioassay validation to recommended protocols. Critical parameters for UV disinfection units are dependent upon manufacturers' design, lamp selection, tube materials, ballasts, configuration, control systems, and associated appurtenances. Spare parts and materials need to be kept on-site; manufacturers can provide recommendations. For additional details on critical design and operational parameters and UV equipment refer to Environment Canada's *UV Guidance Manual for Municipal Wastewater Treatment Plants in Canada* (2002).

14.4.2 UV Disinfection Equipment

UV disinfection systems are proprietary and the designer should consult vendors for specific design details, such as lamp module design, cleaning systems, safety requirements and spare part needs.

14.4.2.1 Lamps

Ultraviolet light is produced in disinfection systems by electrically powered mercury vapour lamps. The lamps are characterized by both their operating pressure and their output level. The three major types of lamps that are available are:

- Low pressure/low intensity (LP/LI);
- Low pressure/high intensity (LP/HI); and
- Medium pressure/high intensity (MP/HI).

The decision on which lamp to use for a specific STP is dependent upon a number of factors. One major factor is the plant's design flow rate. For example, smaller plants are more likely to use low pressure/low intensity lamps than larger plants.

14.4.2.2 Low Pressure/Low Intensity Lamps

Low pressure/low intensity (LP/LI) lamps emit UV radiation that is essentially monochromatic at a wavelength of 253.7 nm, which is within the optimal germicidal range for UV light. The filling of low pressure/low intensity lamps is a mixture of mercury and an inert gas such as Argon. The pressure is quite low (10^{-3} to 10^{-2} mm Hg or 0.0 Pa; i.e., vacuum) as is the operating temperature which is approximately 40 to 50 °C (104 to 122 °F). Most of the mercury remains in liquid form during the operation of the lamp with only a small fraction being vapourized. In disinfection systems two standard lamp lengths have commonly been used, 0.9 m (36 in) and 1.6 m (64 in).

The UV output of LP/LI lamps is relatively low. As a result, UV systems using these lamps require a large number of lamps. The input power for one of the 1.6 m lamps is approximately 75 W, not including the ballast contribution. Including the ballast contribution, the power consumption is approximately 80 to 85 W per lamp. The output of the 1.6 m lamps at the germicidal wavelength of 253.7 nm is 26.7 W. This translates to a germicidal efficiency in the range of 35 to 40 percent. Effective lamp life ranges between 8000 and 13,000 hours. The UV output of the lamps decreases with age.

In most systems using the traditional LP/LI lamps, the lamps are either on or off. Their power is not modulated although the newer electronic ballasts allow for this possibility.

14.4.2.3 Low Pressure/High Intensity Lamps

Low pressure/high intensity (LP/HI) lamps operate within the same pressure range as conventional low pressure lamps [10^{-3} to 10^{-2} mm Hg (0 Pa); i.e., vacuum] and also produce essentially monochromatic radiation at a wavelength of 253.7 nm. The operating temperature for these lamps is 90 to 250°C (194 to 482 °F). The major difference between the low- and high-intensity lamps is that the LP/HI lamps have a much higher power output than the LP/LI lamps and thus requiring fewer lamps to achieve the same level of disinfection.

Power ratings for LP/HI lamps range from 190 to 1620 W with a germicidal efficiency ranging from 20 to 30 percent. This translates to a germicidal UV output per lamp ranging from approximately 40 to 500 W (considering the entire range of possible lamp power ratings). LP/HI lamps are less efficient than LP/LI lamps, but are more efficient than MP/HI lamps. Most low pressure/high intensity systems allow the lamp power to be modulated and have automatic cleaning systems.

The higher power levels used in these lamps, compared to the traditional lamps, are possible due to changes in the size and shape of the lamps, and in the composition of the gas and mercury mixture in the lamps. Some manufacturers use an amalgam of bismuth, indium, and mercury in their lamps that helps maintain the ideal mercury vapour pressure over a wide range of lamp operating temperatures.

The effective life of LP/HI UV lamps ranges from 5000 to 12,000 hours.

14.4.2.4 Medium Pressure/High Intensity Lamps

Medium pressure/high intensity (MP/HI) lamps operate on the same principle as the low pressure lamps. The major difference is that they operate at pressures of 102 to 104 mm Hg (torr) and temperatures of 600 to 800 °C (1112 to 1472 °F). In the MP/HI lamps the mercury is completely vapourized and the pressure is set by the amount of mercury added during the manufacturing process. As with the low-pressure lamps, an inert gas such as argon is also present in the lamp.

The lamps range from 50 to 75 cm (20 to 30 in) in length with a diameter of approximately 2.5 cm (1 in). Quartz sleeves are provided around the lamps for protection and insulation. Automatic cleaning mechanisms are provided for these lamps to minimize fouling which can be of more importance with these lamps due to the increased scaling potential at the higher operating temperatures. The cleaning mechanisms can be mechanical wipers or mechanical wipers augmented with chemical cleaning. Systems with chemical cleaning use an acid solution contained in a chamber incorporated into the mechanical wiper.

Medium pressure lamps have much higher input and output power levels than LP/LI lamps so that fewer lamps are required to achieve the same level of disinfection. Unlike low pressure lamps, the operating temperature of these lamps is not affected by the sewage temperature.

The input power varies with UV supplier. The currently available lamps range from 1,250 to 5,000 W. Power modulation is a common feature of systems with medium-pressure lamps.

The disadvantage of the medium pressure lamps is that they are less efficient than the low-pressure lamps. They convert about the same percentage of their input power to radiation but this radiation is polychromatic with wavelengths ranging from 180 to 1,370 nm. Only a small portion of this radiation is in the optimal germicidal range. Germicidal efficiencies for medium pressure lamps range from 7 to 15 percent. This translates to a germicidal UV output per lamp ranging from 87.5 to 750 W (considering the entire range of possible lamp power ratings).

Effective lamp life for medium pressure lamps ranges from 5,000 to 8,000 hours. As with the low pressure lamps, the lamp output decreases with age.

14.4.2.5 Ballasts

Ballasts are transformers that regulate the current to the UV lamps and stabilize the light output. They also provide sufficient voltage to start the lamp. Without a ballast, the current in a mercury vapour lamp keeps increasing until the lamp overheats and destroys itself.

Generally in older UV systems, electromagnetic ballasts were used. The traditional low pressure/low intensity lamps are instant start lamps. In systems using these lamps, each electromagnetic ballast powers two lamps. A step-up transformer is used to create the starting arc without pre-heating the electrodes.

Electromagnetic ballasts are reliable but have many disadvantages. The major disadvantage is that they are inefficient and lose much of their input energy as heat. As a result, they require cooling systems and temperature monitors. Other problems with these ballasts are their noise, size, weight, and inability to modulate the power supply to the lamps.

Most new UV systems use electronic ballasts. These ballasts are smaller, lighter, and more energy efficient than electromagnetic ballasts. They also allow for power modulation so that the lamps can be dimmed or brightened. In this case rapid start lamps with continuously heated electrodes are used. Electronic ballasts can supply one or more lamps each.

Although electronic ballasts produce much less heat than electromagnetic ballasts, they still produce enough heat to cause problems unless adequate cooling is provided. Most UV systems, particularly medium pressure systems, are equipped with ballast cooling systems such as fans or heat exchangers with a circulating coolant such as propylene glycol. Some manufacturers use submerged ballasts in their systems that are cooled by the sewage flow. Besides being self-cooled, submerged ballasts offer the advantage of reducing the size of the cabinets needed to house the UV electrical equipment.

14.4.2.6 Reactors

Ultraviolet disinfection reactors are available in both open channel and closed chamber configurations. Systems that use low pressure lamps (both low and high intensity) have open channel reactors while systems with medium pressure lamps have enclosed reaction chambers. The medium pressure systems can be installed in an open channel, but its lamps are contained in an enclosed reaction chamber. Alternatively, medium-pressure systems can be contained in a closed pipe. Spare parts requirements are dependent on the system used and a plant's requirements for operation and maintenance.

Open Channel Reactors

In open channel reactors, the lamps can be arranged horizontally and parallel to the sewage flow or vertically. Horizontal lamp, open channel configurations are the most common UV disinfection installations in municipal STPs. Horizontal systems consist of modules of lamps that are hung together side-by-side in banks in an open channel. The modules span the width of the channel to form a bank. Each module consists of a metal support frame that contains a number of evenly spaced lamps enclosed in quartz sleeves. The modules typically contain 8 or 16 lamps, but may contain fewer lamps depending on the size of the application and lamp type.

Vertical lamp systems for open channel installations were introduced as alternatives to horizontal systems in the late 1980s. Vertical lamp modules consist of an open rectangular frame that rests on the bottom of the channel in an upright position. The lamps are oriented so that they are perpendicular to the floor of the channel. A vertical module typically contains 40 lamps that are arranged in a staggered 8 by 5 array.

Closed Channel Reactors

Closed channel reactors are used in systems with medium pressure lamps. There are two basic designs:

- Horizontal and parallel to flow lamps enclosed in a reaction chamber that is housed in an open channel; and
- Horizontal and perpendicular to flow lamps mounted in a closed chamber that is connected to flanged pipes.

14.4.2.7 Cleaning Mechanisms

In sewage treatment, degradation of sleeve transmittance by effluent is inevitable. This is a direct result of precipitation and fouling of the quartz sleeves due to the presence of iron, calcium, aluminum, manganese and other organic and inorganic matter in the sewage effluent. The impact of this build-up of film and debris has a pronounced effect on the amount of UV energy that is transmitted into the surrounding sewage. Therefore, efficient and effective disinfection performance is dependent on maintaining clean sleeves that ensure maximum delivery of UV energy to the sewage effluent. To minimize degradation of sleeve transmittance various cleaning approaches are available. These include out-of-channel cleaning tanks, manual wiping and acid recirculation systems and/or automatic wiper systems.

14.4.3 Design Considerations

The following factors need to be taken into consideration when a UV system is being designed for disinfection of sewage effluent. The designer needs to provide this information to the UV manufacturer because each UV system is designed on an individual basis. These considerations include:

- Type of treatment processes used upstream of the disinfection stage;
- Type of chemicals added to upstream treatment processes;
- UV transmission or absorbance (assume 60 percent UV transmittance for secondary effluent; higher for a tertiary effluent);
- Suspended solids (concentration and nature of particulate matter);
- Available head, flow rate and hydraulics;
- Iron content of influent to UV process;
- Hardness and alkalinity;
- Sewage sources and sewage effluent characteristics;
- Reactor hydraulic considerations; and
- Disinfection requirements (*Section 14.1.1-Disinfection Requirements*).

Open channel designs with modular UV disinfection units that can be removed from the flow are often recommended. At least two banks in series needs to be provided in each channel for disinfection reliability and to ensure uninterrupted service during lamp sleeve/tube cleaning or other required maintenance. An automatic switchover should be provided to switch to the standby UV bank operation upon a failure of the duty bank. Operator safety (electrical hazards and UV exposure) and tube cleaning frequency should be considered. Manufacturer's manuals should be referred to for details and requirements. The hydraulic properties of the system should be designed to simulate plug flow conditions under the full operating flow range. In addition, a positive means of water level control should be provided to achieve the necessary exposure time.

To ensure adequate UV irradiation, the maximum liquid surface elevation within the UV reactor basin should not be any greater than the manufacturer's recommendations or 25 to 50 mm (1 to 2 in) above the UV lamps, or there will be potential for inadequate disinfection due to short-circuiting. The minimum water surface elevation within the UV reactor basin should not expose the UV lamps to air or there will be potential for burning the medium-pressure UV lamps or having material dry on the surface of the quartz sleeves of the low-pressure lamps. Because of the water surface constraints, the maximum fluctuation of liquid surface elevation should be limited to 50 mm (2 in) over the range of flow conditions. Devices typically used to maintain the water surface elevations are counterbalanced flap gates, serpentine weirs or control gates. Serpentine weirs are the easiest device to operate and are recommended for smaller facilities or upstream batch treatment processes [e.g. Sequencing Batch Reactors (SBRs)]. In SBRs with periodic discharges, consideration needs to be given to flow equalization in order to be able to maintain the UV system in continuous operation. Otherwise, UV designs need to be based on short-term peak flow through UV units during periodic SBR decanting operations.

Closed channel/chamber or pressurized systems are less commonly used in sewage effluent applications. Closed channel systems are well suited for below ground installations where the effluent is under pressure and should be confined in a closed vessel.

The UV process is most effective for secondary effluent or better quality; lower quality effluents will require a higher UV dosage. The UV dosage should be based on the design peak hourly flow. As a general guide, in system sizing for an activated sludge effluent with the preceding characteristics, a UV dosage not less than 30 $(mW{\cdot}s)/cm^2$ may be used after adjustments for maximum tube fouling, lamp output reduction at the end of lamp life (for example 8760 hours for some systems), and other energy absorption losses. These are only general guidelines and the UV dosage, lamp life and fouling are all sewage effluent and system dependent.

Sizing UV disinfection systems is conservative in that it is assumed that there will be a simultaneous occurrence of the worst case conditions for the input variables. Input variables required include maximum, minimum, and average flow; minimum UV transmission (filtered and unfiltered); maximum TSS concentration; maximum indicator organism log reduction; maximum quartz sleeve fouling; minimum UV lamp output; and allowances for potential photo reactivation of microorganisms.

Current procedure is for the UV system to be designed to deliver the required UV dose at peak flow, in effluent with a UV transmission stated at the end of lamp life (EOLL) after reductions for quartz sleeve fouling. The basis for evaluating the UV dose delivered by the UV system will be the independent third-party bioassay. Bioassay validation methodology should follow protocols described in National Water Research Institute (NWRI) *Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse* (May 2003) and/or applicable sections of the U.S. EPA *Design Manual – Municipal Wastewater Disinfection* (EPA/625/1-86/021).

Refer to design manuals (i.e., U.S. EPA and others) for EOLL and fouling factors for design.

An alarm system needs to be provided to separately indicate lamp failure and low UV intensity. A UV intensity meter should be used for this application as part of continuous monitoring.

Consideration needs to be made for operation and maintenance requirements of the UV equipment, especially the UV lamps. Sufficient area and lifting devices need to be provided to accommodate maintenance and changing of UV lamps. For smaller systems, if automatic cleaning is not provided, the provision of a cleaning tank external to the channel(s) with all required needs (i.e., potable water and electrical power) should be provided.

14.4.4 Safety

Most of the related safety issues revolve around electrical hazards or exposure to UV irradiation when the lamps are not submerged. Equipment should be provided with safety interlocks that shut down the UV banks or modules if moved out of their position or the liquid level drops below the top row of lamps in a horizontal system or exposes the top portion of the UV lamps in a vertical system. The vertical system may include light shields that allow a small portion of the tops of the lamps to be exposed to air without being a hazard. Ground fault interruption circuitry or other UL Approved electrical safety features should be provided. Whenever low-pressure UV lamps are to be handled, personnel should be equipped with face safety shields rated to absorb light with wavelengths ranging from 200 to 400 nm and all exposed skin should be covered. Safety shields for medium-pressure UV lamps should be rated to absorb light with wavelengths ranging from 100 to 900 nm and all exposed skin should be covered. An arc welder's mask should be used with medium-pressure UV lamps.

14.5 OZONATION

Design standards, operating data, and experience for this process are not well established. Therefore, design of these systems should be based upon experience at similar full-scale installations or thoroughly documented prototype testing with the particular sewage at site-specific conditions.

The main advantages of ozonation over chlorination include its capability to increase the dissolved oxygen of the effluent and the absence of potentially carcinogenic disinfection byproducts. Also, ozone is capable of destroying a wide spectrum of viruses and bacteria and is not as susceptible to the effects of ammonia and pH as chlorine. Problems associated with transportation of toxic chemicals are eliminated since ozone has to be generated on-site.

The main disadvantages of ozonation compared to chlorination are higher capital costs and greater operational complexity. Ozone demand is high for sewage effluents with high iron content; if the treatment plant influent has a large industrial contribution, ozone disinfection is less cost effective.

Design of an ozone disinfection process involves sizing the ozone generation equipment and contact basins to meet the disinfection requirements over the anticipated range of operating conditions. The design requirements for ozonation systems should be based on pilot testing or similar full-scale installations.

Ozone dosage is described as either the applied dosage or transferred dosage, the two being related by the ozone transfer efficiency. The applied ozone dosage is a function of the ozone production rate and the sewage flow rate. The transferred dosage requirement is determined by the applicable effluent standard and the COD content of sewage effluent to be disinfected. For tertiary-filtered non-nitrified secondary effluent, about 12 to 15 mg/L of transferred ozone dosage is used to ensure that the monthly geometric mean density of *E. coli* does not exceed 200 organisms per 100 millilitres in the effluent discharge from the sewage treatment plant, while for filtered nitrified effluent, the dosage used ranges from 3 to 5 mg/L.

The contact time required to achieve a specified effluent disinfection requirement depends on sewage characteristics and applied ozone dosages. Contact times ranging from 2 to 10 minutes have been reported.

Because ozone is a toxic gas, excess ozone should be removed from the contact basin off-gas stream prior to venting, recycle, or reuse of the off-gas. Off-gas ozone disposal could be accomplished through reinjection, chemical reduction, dilution, thermal destruction, catalytic destruction, and/or activated carbon adsorption.

CHAPTER 15

SUPPLEMENTAL TREATMENT PROCESSES

CHAPTER 15

SUPPLEMENTAL TREATMENT PROCESSES

This chapter describes supplemental sewage treatment processes such as those used for phosphorus control, tertiary or quaternary treatment and alternative disposal of treated sewage effluent. Physical/chemical phosphorus and ammonia removal, effluent filtration, microscreening, membrane systems, tertiary clarifiers, natural systems, persistent organics removal and land application of treated final effluent are presented in this chapter. Some processes presented in this chapter may not be common (or applicable for use) in Ontario, but have been included for completeness. Care should be taken in applying processes that are not commonly used in Ontario. A summary of the design criteria for some of the supplemental treatment processes is provided in Appendix V, which should be used in conjunction with the details in this chapter.

15.1 PHOSPHORUS REMOVAL BY CHEMICAL TREATMENT

15.1.1 General

15.1.1.1 Method

The reduction of total phosphorus (TP) concentration in the effluent to 1 mg/L (monthly average basis) can consistently be achieved by chemical coagulation and sedimentation or by biological phosphorus removal (BPR) processes, see *Section 12.4.8 - Biological Nutrient Removal*). TP concentrations in the effluent lower than 1.0 mg/L, down to 0.5 mg/L (monthly average basis), have been demonstrated at some secondary treatment plants without filtration.

Batch chemical dosing of seasonal retention lagoon systems before discharge may be able to achieve 0.5 mg/L effluent TP levels.

Addition of aluminum salts, iron salts or lime (less common) may be used for the chemical removal of soluble phosphorus. The phosphorus reacts with the aluminum, ferrous/ferric or calcium ions to form insoluble compounds. Those insoluble compounds may be flocculated with or without the addition of a coagulant aid such as a polyelectrolyte to facilitate separation by sedimentation, or sedimentation followed by filtration.

15.1.1.2 Design Basis

Laboratory, pilot or full scale studies of various chemicals, feed points and treatment processes are recommended for existing plant facilities to determine the achievable performance level, cost-effective design criteria, ranges of required chemical doses and chemical addition points.

The selection of a treatment process and chemical dosage for a new facility should be based on such factors as influent sewage characteristics, the proposed chemical and effluent requirements.

Systems should be designed with sufficient flexibility to allow for several operational adjustments in chemical feed location, dosing rates and alternate chemical compounds.

15.1.2 Process Requirements

15.1.2.1 Dosage

The design chemical dosage should be based on the amount needed to react with the phosphorus in the sewage to meet the required removal and the amount required due to inefficiencies in mixing or dispersion. Excessive chemical dosages should be avoided.

With secondary treatment plants, the chemical dosage requirements for either alum or ferric chloride have been found to be least when the addition of chemical is made to the aeration tank effluent. Dosing to the aeration tank influent requires as much as 35 percent higher dosage rates.

Typical dosing rates of commercial grade chemicals needed for total phosphorus reduction to the 1.0 mg/L level are:

- Alum 110 to 225 mg/L as $Al_2(SO_4)_3 \cdot 14H_2O$;
- Ferric salts 6 to 30 mg/L as Fe; and
- Lime 40 to 400 mg/L as $Ca(OH)_2$.

15.1.2.2 Chemical Selection

The choice of aluminum salts, iron salts or lime should be based on the sewage characteristics, chemical availability and handling, sludge processing and disposal methods and the economics of the total system.

When lime is used, it may be necessary to neutralize the resultant higher pH prior to subsequent treatment in biological processes or prior to discharge in those flow schemes where lime treatment is the final step in the treatment process. Problems associated with lime usage, handling and sludge production and dewatering should be recognized and evaluated.

15.1.2.3 Chemical Feed Points

Selection of chemical feed points should include consideration of the chemicals used in the process, necessary reaction times (3 to 5 minutes) between chemical and polyelectrolyte additions and the sewage treatment processes utilized.

A number of addition points should be made to provide flexibility and to improve phosphorus removal, reduce loadings to biological process and/or reduce chemical usage. Common chemical addition points for sewage treatment plants (STPs) are:

- **Pre-Precipitation** - the addition of chemical to the pretreated (i.e., screened and degritted) raw sewage prior to primary clarification. Pre-precipitation on its own may meet final effluent phosphorus goals and has the advantage of reducing the organic loading to the biological process by increasing BOD_5 removal through the primary treatment stage. The process can be enhanced by the addition of polymer at low dosages (e.g. 0.5 to 1.0 mg/L) although care should be taken to ensure adequate phosphorus remains for the biological process. Mixing is critical and supplemental mixing may be required;
- **Simultaneous Precipitation** - the addition of chemical to the biological process is the most common addition point for chemical precipitation for phosphorus removal. Addition is generally to the end of the aeration tank prior to the effluent weir or outlet to make use of the aeration in the tank for mixing the chemical with the mixed liquor suspended solids (MLSS);
- **Dual Point Addition** - the addition of chemical to two locations in the STP, generally pre- and simultaneous precipitation (i.e., to the pretreated sewage and to the biological process). Dual point addition allows for enhanced control of the chemical precipitation process and the ability to meet low effluent total phosphorus limits. Phosphorus levels can potentially be reduced to 0.3 mg/L through the use of dual-point addition over single point addition of chemical; and
- **Post-Precipitation** - the addition of chemical after the secondary clarifiers but before a tertiary treatment step, such as filtration. Post-precipitation allows for low levels of TP to be achieved, with no risk to biological processes. It has been demonstrated that drum filters, sand filters or ballasted clarifiers can together with post-precipitation produce 0.1 mg/L total phosphorus on a monthly average basis.

15.1.2.4 Flash Mixing and Flocculation

Each chemical should be mixed rapidly and uniformly with the sewage stream. The mixing action in an aeration tank can accomplish adequate mixing. Where separate mixing tanks are provided, they should be equipped with mechanical mixing devices. The detention period should be at least 30 seconds.

The particle size of the precipitate formed by chemical treatment may be very small. Consideration should be given in the process design to the addition of synthetic polyelectrolytes to aid settling. The flocculation equipment should be adjustable in order to obtain optimum floc growth, control deposition of solids and prevent floc destruction.

Flocculation tanks are often used prior to primary clarification or for non-aerated biological process (e.g. rotating biological contactors or tricking filters) or where adequate mixing is not provided by the processes. Flocculation tanks are also used when both a coagulant and polymer are being added.

15.1.2.5 Solids Separation and Filtration

The velocity through pipes or conduits to sedimentation tanks should not exceed 0.5 m/s (1.5 ft/s) in order to minimize floc destruction. Entrance works to sedimentation tanks should also be designed to minimize floc shear.

Sedimentation tank design should be in accordance with criteria outlined in *Chapter 11 - Primary Sedimentation* and *Chapter 13 - Secondary Sedimentation*. For design of the sludge handling system, special consideration should be given to the type and volume of sludge generated by the phosphorus removal process.

Effluent filtration should be considered where effluent phosphorus concentrations of less than 0.5 mg/L level need to be achieved.

15.1.3 Feed Systems

15.1.3.1 Location

All liquid chemical mixing and feed installations should be installed on corrosion resistant pedestals and elevated above the highest liquid level anticipated during emergency conditions.

The chemical feed equipment should be designed to meet the maximum dosage requirements for the design conditions.

Lime feed equipment should be located so as to minimize the length of slurry conduits. All slurry conduits should be accessible for cleaning.

15.1.3.2 Liquid Chemical Feed System

Liquid chemical feed pumps should be of the positive displacement type with variable feed rate. Pumps should be selected to feed the full range of chemical quantities required for the phosphorus mass loading conditions anticipated with the largest unit out of service. Consideration should be given to systems, including pumps and piping, that can feed either aluminum or iron compounds to provide flexibility.

Screens and valves should be provided on the chemical feed pump suction lines.

An air break or anti-siphon device should be provided where the chemical solution stream discharges to the transport water stream to prevent an induction effect resulting in overfeed.

The designer should consider providing flow pacing equipment to match chemical feed rates with sewage flow rates.

15.1.3.3 Dry Chemical Feed System

Each dry chemical feeder should be equipped with a dissolver that is capable of providing a minimum 5-minute retention time at the maximum feed rate.

Polyelectrolyte feed installations should be equipped with two solution vessels and transfer piping for solution make-up and daily operation.

Make-up tanks should be provided with an eductor funnel or other appropriate arrangement for wetting the polymer during the preparation of the stock feed solution. Adequate mixing should be provided by a large-diameter low-speed mixer.

15.1.4 Storage Facilities

Storage facilities should be sufficient to ensure that an adequate supply of the chemical is available at all times. Storage volume requirements will depend on size of shipment, length of delivery time and process requirements. Storage for a minimum supply of 10 days should be provided.

The liquid chemical storage tank and tank fill connections should be located within a containment structure having a capacity exceeding the total volume of all storage vessels. Valves on discharge lines should be located adjacent to the storage tank and within the containment structure.

Auxiliary facilities, including pumps and controls, within the containment area should be located above the highest anticipated liquid level. Containment areas should be sloped to a sump area and should not contain floor drains.

Bag storage should be located near the solution make-up point to avoid unnecessary transportation and housekeeping problems.

Platforms, stairs and railings should be provided, as necessary, to afford convenient and safe access to all fill connections, storage tanks and measuring devices.

Storage tanks should have access provided to facilitate cleaning.

15.1.5 Other Requirements

The chemical feed equipment and storage facilities should be constructed of materials resistant to the chemicals normally used for phosphorus removal.

Precautions should be taken to prevent chemical storage tanks and feed lines from reaching temperatures likely to result in freezing or chemical crystallization at the concentrations employed. A heated enclosure or insulation may be required. Consideration should also be given to humidity and dust control in all chemical feed room areas.

Piping should be accessible and installed with plugging wyes, tees or crosses with removable plugs at changes in direction to facilitate cleaning.

Above-bottom draw off from chemical storage or feed tanks should be provided to avoid withdrawal of settled solids into the feed system. A bottom drain should also be installed for periodic removal of accumulated settled solids. Provisions should be made in the fill lines to prevent back siphonage of chemical tank contents.

The chemical handling facilities should meet the appropriate safety and hazardous chemical handling facilities requirements (*Section 20.5 - Operator Safety*).

Consideration should be given to the type and additional *capacity* of the sludge handling facilities needed when chemicals are added at a sewage treatment plant (STP).

15.2 HIGH RATE EFFLUENT FILTRATION

15.2.1 General

Granular media filters may be used as an advanced treatment process for the removal of residual TSS and TP from secondary effluent. Filters may be necessary where effluent concentrations of less than 15 mg/L of TSS and/or 0.5 mg/L of TP need to be achieved. A pretreatment process such as chemical coagulation and sedimentation or other acceptable process should precede the granular media filter units where effluent suspended solids requirements are less than 10 mg/L.

With pretreatment of secondary effluent and conservative filtration system design, effluent quality of 5 mg/L $CBOD_5$, 5 mg/L TSS and 0.1 mg/L total phosphorus can be achieved.

If chlorine disinfection is used at an STP, effluent filtration should precede the chlorine contact chamber to minimize chlorine usage, to allow more effective disinfection and to minimize the production of chloro-organic compounds.

To periodically remove excessive biological growths and grease accumulations from the filter media, a chlorine application point should be provided upstream of the filtration system. Chlorine would only be dosed as necessary at this location.

Influent flow weirs are recommended to avoid flow split issues and non-uniform fouling of filters

Care should be given in designing pipes or conduits ahead of filter units, if applicable, to minimize shearing of floc particles. Consideration should be given in the plant design to provide flow equalization facilities to moderate filter influent quality and quantity.

15.2.2 Filter Types

There are various types of effluent filtration systems including: single, dual and mixed-media systems; shallow and deep bed systems; upflow and downflow filters; gravity and pressure systems; continuous and discontinuous operation filters; cloth or fabric media filters; manual and automatic backwash filters; and slow sand filters.

Pressure filters should be provided with ready and convenient access to the media for inspection and cleaning. Pressure filters should not be used where abnormal quantities of grease or similar solids that may result in filter plugging are expected. Pressure filters are less common in Ontario, but might be considered if building space is limited.

Factors to consider when choosing between the different filtration systems include the following:

- The effluent quality requirements;
- The energy requirements of the systems (head requirements);
- The media types, sizes, solids capture capacities and treatment efficiencies of the systems;
- The backwashing systems, including type, backwash rate, backwash volume and effect on *sewage works*; and
- The installed capital and expected operation and maintenance costs.

Disk and drum filters provide a large filter area in a small footprint. Drum filters help to prevent particle fragmentation.

Continuous backwash filters allow for a constant flow to be maintained and eliminate spikes in filtration performance that are generally found after backwash cycles.

15.2.3 Filter Media Selection

Selection of appropriate media type and size will depend on required effluent quality, the type of treatment provided prior to filtration, the filtration rate selected and filter configuration. In dual- or multi-media filters, media size selection should consider compatibility among media. The selection and sizing of the media should be based on demonstrated satisfactory field experience under similar conditions. All media should have a uniformity coefficient of 1.7 or less. The uniformity coefficient, effective size, depth and type of media should be set forth in the specifications.

15.2.4 Filtration Rates

Filtration rates at design peak hourly flows, including backwash flows, should not exceed 2.1 L/(m^2·s) (3 USgpm/ft^2) for shallow bed single media systems and should not exceed 3.3 L/(m^2·s) (4.8 USgpm/ft^2) for deep bed filters. Shallow bed single media filters generally have 0.6 m (2 ft) of media depth; deep or multi media filters generally have 1.2 to 1.8 m (4 to 6 ft) of media depth. If flow equalization is provided, appropriately lower peak flows should be used in order to avoid oversizing of the filter.

The manufacturer's recommended maximum filtration rate should not be exceeded.

Peak solids loading rate should not exceed 51 mg/(m^2·s) [(0.038 lb/(ft^2·hr)] for shallow bed filters and 83 mg/(m^2·s) [(0.061 lb/(ft^2·hr)] for deep bed filters.

Total filter area should be provided in two or more units and the filtration rate should be calculated on the total available filter area with one unit out of service.

15.2.5 Backwash

Air scour or mechanical agitation systems to improve backwash effectiveness are recommended. The backwash rate should be adequate to fluidize and expand each media layer a minimum of 20 percent based on the media selected. Backwash rates should be at least 10 L/(m^2·s) (14.7 USgpm/ft^2) or whatever rate is necessary to achieve at least 20 percent bed expansion. The backwash system should be capable of providing variable backwash rates. Minimum and maximum backwash rates should be based on demonstrated satisfactory field experience under similar conditions. The design should provide for a minimum backwash period of 10 minutes.

Pumps for backwashing filter units should be sized and interconnected to provide the required backwash rate to any filter with the largest pump out of service. Filtered water from the clear well or chlorine tank, if available, should be used as the source of backwash water. Backwash waters should be returned to the primary sedimentation tanks or to the effluent end of the aeration tanks (if there is no primary sedimentation stage).

The rate of return of waste filter backwash water to treatment units should be controlled so that the rate does not exceed 15 percent of the design average daily flow rate to the treatment unit. The hydraulic and organic load from waste backwash water should be considered in the overall design of the STP. Surge tanks should have a minimum capacity of two backwash volumes, although additional capacity should be considered to allow for operational flexibility. Where waste backwash water is returned for treatment by pumping, adequate pumping capacity should be provided with the largest unit out of service.

Total backwash water storage capacity provided in an effluent clearwell or other unit should equal or exceed the volume required for two complete backwash cycles.

15.2.6 Filter Appurtenances

The filters should be equipped with washwater troughs, surface wash or air scouring equipment, means of measurement and positive control of the backwash rate, equipment for measuring filter head loss, positive means of shutting off flow to a filter being backwashed and filter influent and effluent sampling points. If automatic controls are provided, there should be a manual override for operating equipment, including each individual valve essential to the filter operation. The underdrain system should be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water. If air is to be used for filter backwash, separate backwash blower(s) should be provided. The designer should provide for periodic chlorination of the filter influent or backwash water to control slime growth. When chemical disinfection is not provided at the plant, manual dosage of chlorine compounds is an option.

15.2.7 Access and Housing

Each filter unit should be designed and installed so that there is ready and convenient access to all components and the media surface for inspection and maintenance without taking other units out of service. Housing for filter units should be provided. The housing should be constructed of suitable corrosion-resistant materials. All controls should be enclosed and the structure housing filter, controls and equipment should be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

15.3 MICROSCREENING

15.3.1 General

Microscreening units may be used following a biological treatment process for the removal of residual suspended solids. Microscreening should not be considered as an alternative to granular media filters. Low efficiencies in treating secondary effluents have been reported. Microscreening has been effective in removing coarse and filamentous types of algae and other suspended solids in some instances. Microscreens have been used in place of clarifiers to polish effluent from low-rate trickling filters where the solids are generally low in concentration and well flocculated.

Selection of this unit process should be carefully evaluated and this review should consider final effluent requirements, the preceding biological treatment process and anticipated consistency of the biological process to provide a high quality effluent.

15.3.1.1 Design Considerations

Pilot plant testing with existing secondary effluent is encouraged. Where pilot studies so indicate, a pretreatment process such as chemical coagulation and sedimentation should be provided. Care should be taken in the selection of pumping equipment ahead of microscreens to minimize the shearing of floc particles. The process design should include flow equalization facilities to moderate microscreen influent quality and quantity.

The following items should be considered:

- Automatic control of the drum microscreen rotational speed and/or the backwash rate via a head loss control system;
- Appropriate measures to control biological slime growth on the screen;
- A minimum of two independent units; and
- The hydraulic and organic loading from the waste backwash water.

A supply of critical spare parts should be provided and maintained. All units and controls should be enclosed in a heated and ventilated structure with adequate working space for maintenance.

The microfabric should be a material demonstrated to be durable through long-term performance data. The aperture size should be selected considering required removal efficiencies, normally ranging from 20 to 35 microns. The use of pilot plant testing for aperture size selection is recommended.

15.3.2 Screening Rate

The screening rate should be selected to be compatible with available pilot plant test results and selected screen aperture size, but should not exceed 3.4 $L/(m^2 \cdot s)$[0.083 US $gal/(ft^2 \cdot s)$] of effective screen area based on the design peak instantaneous flow rate applied to the units. The effective screen area should be considered as the submerged screen surface area less the area of screen blocked by structural supports and fasteners. The screening rate should be that applied to the units with one unit out of service.

The hydraulic design should provide a head loss through the screen no greater than 76 mm (3 in) at design average daily flow and 15.2 cm (6 in) at normally expected design peak instantaneous flows. Under no circumstances should head loss through the screen exceed 610 mm (24 in).

15.3.3 Backwash

All waste backwash water generated by the microscreening operation should be recycled for treatment. The backwash volume and pressure should be adequate to ensure maintenance of fabric cleanliness and flow capacity. Equipment for backwash of at least 1.65 L/(m·s)[0.133 US gal/(ft·s)] of screen length and 420 kPa (60 psi), should be provided. Backwash water should be supplied continuously by multiple pumps, including one standby, and should be obtained from microscreened effluent. The rate of return of waste backwash water to treatment units should be controlled such that the rate does not exceed 15 percent of the design average daily flow rate to the STP. The hydraulic and organic load from waste backwash water should be considered in the overall design of the treatment plant. Where waste backwash is returned for treatment by pumping, adequate pumping capacity should be provided with the largest unit out of service. Provisions should be made for measuring backwash flow.

15.4 MEMBRANES

15.4.1 General

Membrane separation processes such as ultrafiltration and microfiltration can effectively remove particulate and some colloidal matter, producing a highly polished effluent stream (i.e., *permeate*). Ultrafiltration can also remove certain dissolved solids, depending on the molecular weight cut-off (MWCO) rating of the membrane. Membrane separation is a pressure-driven physical filtration process capable of capturing material in the 0.002 to 0.2 micron range or larger in the case of ultrafiltration and the 0.1 to 2 micron range or larger for microfiltration, depending on the MWCO and pore size rating for the respective membrane types. Ultrafiltration processes also have the significant benefit of decreasing residual BOD_5, numbers of cysts, bacteria and viruses, and enhancing the performance of subsequent disinfection processes.

15.4.2 Design Considerations

The following items should be considered during design of membrane-based polishing processes:

- The applied pressure or vacuum is normally below 690 kPa (100 psi), typically between 140 to 690 kPa (20 to 100 psi) for ultrafiltration and 35 to 210 kPa (5 to 30 psi) for microfiltration;
- Liquid velocities of 0.9 to 3.0 m/s (3 to 10 ft/s) parallel to the surface of the membrane (i.e., cross-flow filtration) helps to scour membrane surfaces and provide a more stable *flux* through the membrane;
- Upstream pretreatment systems typically precede ultrafiltration systems to remove coarse solids and to enhance run time;
- Influent TSS levels below 15 mg/L are preferred for ultrafiltration units to extend run time;
- Pilot testing should be used for membrane selection and to provide site-specific operating data;
- System redundancy should be provided to allow *membrane backwashing* and membrane replacement;
- Backwash flows should have a surge tank to dissipate the hydraulic and solids impacts on downstream components; and
- Ultrafiltration systems should be designed with a minimum initial flux rate of 0.73 m/d (18 USgpd/ft^2) and a minimum final flux rate of 0.20 m/d (5 USgpd/ft^2).

15.4.3 Equipment and Appurtenances

The following appurtenant features should be considered for inclusion in the system:

- Redundancy for feed, backwash and waste pumps should be provided, considering that the largest unit is out of service for each pumping system;
- Membrane support systems should provide uniform backing of membranes and uniform flux rates over the unit; and
- Multiple modular units are desired from an operating and cost perspective. Adequate redundancy should account for the largest unit being out of service and the other units operating at a flux level of 50 percent of the membrane's useful life.

15.5 HIGH RATE CLARIFICATION

Ballasted flocculation, also known as high rate clarification, is a physical/chemical treatment process that uses continuously recycled media and a variety of additives to improve the settling properties of suspended solids through improved floc bridging. The objective of this process is to form microfloc particles with a specific gravity of greater than 2. Faster floc formation and decreased particle settling time allow treatment of flows at a significantly higher rate than with traditional sedimentation processes.

Ballasted flocculation units function through the addition of a coagulant or polymer and a ballast material such as microsand (a microcarrier or chemically enhanced sludge can be used). When combined with chemical addition, this ballast material can reduce coagulation-sedimentation time. Ballasted flocculation units have operated with overflow rates of 815 to 3260 L/(m^2·min) (20 to 80 USgpm/ft^2) while achieving total suspended solids removal of 80 to 95 percent. The compact size of ballasted flocculation units makes them particularly attractive for retrofit and high rate applications. This technology has been applied both within traditional treatment trains, as a parallel treatment train in new or existing sewage works, and as overflow treatment for peak wet weather flow.

Applications of ballasted flocculation include:

- Enhanced primary clarification;
- Enhanced secondary clarification following attached and suspended growth biological processes; and
- Combined sewer overflow (CSO) and sanitary sewer overflow (SSO) treatment.

Major advantages for both new and upgraded treatment operations include:

- The reduced surface area of the clarifiers minimizes short-circuiting and flow patterns caused by wind and freezing;
- Systems using ballasted flocculation can treat a wider range of flows without reducing removal efficiencies; and
- Ballasted flocculation systems reduce the amount of coagulant used or improve settling compared to traditional systems for comparable chemical usage.

Some disadvantages of ballasted flocculation systems include:

- They require more operator judgment and more complex instrumentation and controls than traditional processes; and
- Pumps may be adversely affected by ballast material recycle. Lost microsand or microcarrier should be occasionally replaced (except where settled sludge is recycled for use as a microcarrier/ballast).

15.6 AMMONIA REMOVAL BY PHYSICAL/CHEMICAL TREATMENT

The two main physical/chemical treatment methods used in North America for ammonia removal are breakpoint chlorination and air stripping. It should be noted that neither method is common in Ontario.

15.6.1 Breakpoint Chlorination

The breakpoint chlorination process is best suited for removing relatively small quantities of ammonia, less than 5 mg/L total ammonia nitrogen (TAN) and in situations where low residuals of ammonia or total nitrogen are required. In most applications, dechlorination will be required prior to effluent discharge.

The reaction between ammonia and chlorine occurs rapidly and no special design features are necessary, except to provide for complete uniform mixing of chlorine with the final effluent and dechlorination. Good mixing can best be accomplished with in-line mixers or backmix reactors. A minimum contact time of 10 min is recommended.

The sizing of the chlorinator and a feed device is dependent on the ammonia concentration to be treated as well as the degree of treatment that the final effluent is to receive.

If insufficient chlorine is available to reach the breakpoint, no nitrogen will be formed and the chloramines formed ultimately will revert back to ammonia. Provisions should be made to continuously monitor the effluent, following chlorine addition, for free chlorine residual and to pace the chlorine feed device to maintain a set-point free chlorine residual.

Except for final effluents having a high alkalinity, provisions should be made to feed an alkaline chemical to keep the pH of the final effluent in the proper range. A method for measuring and pacing the alkaline chemical feed pump to keep the pH in the desired range should be provided.

15.6.2 Air Stripping

Air stripping of ammonia is most economical if it is preceded by lime coagulation and settling. Approximately 90 percent of the nitrogen in a non-nitrified treated sewage effluent is in the form of ammonia for which the ammonia stripping process may be suitable. The ammonia stripping process is not suitable if preceded by a nitrifying biological process. Mitigation of air quality impacts from substances that are removed by air stripping need to be considered (*Section 3.11 - Emissions of Contaminants to Air*).

Ammonia stripping in a stripping tower cannot be operated at air temperatures less than 0 °C (32 °F) because of freezing within the tower unless the air is preheated. This makes the application of this technology difficult for year-round application in Ontario.

Packing used in ammonia stripping towers may include 10 by 40 mm (0.4 to 1.6 in) wood slats, plastic pipe and a polypropylene grid. No specific packing spacing has been established. Generally, the individual splash should be spaced 40 to 100 mm (1.6 to 4 in) horizontally and 50 to 100 mm (2 to 4 in) vertically. A tighter spacing is used to achieve higher levels of ammonia removal and a more open spacing is used where lower levels of ammonia removal are acceptable. Towers should be designed for a total air headloss of less than 50 to 75 mm (2 to 3 in) of water because of the large volume of air required. Packing depths of 6 to 7.5 m (20 to 25 ft) should be used to minimize power costs.

Allowable hydraulic loading is dependent on the type and spacing of the individual splash bars. Although hydraulic loading rates used in ammonia stripping towers should range from 0.7 to 2.0 $L/(m^2 \cdot s)$ [0.02 to 0.05 US $gal/(ft^2 \cdot s)$], removal efficiency is significantly decreased at loadings in excess of 1.3 $L/(m^2 \cdot s)$ [0.03 US $gal/(ft^2 \cdot s)$]. The hydraulic loading rate should be such that a water droplet is formed at each individual splash bar as the liquid passes through the tower.

Air requirements vary from 2,200 to 3,800 L/s (580 to 1,000 US gal/s) for each 1 L/s (0.26 US gal/s) of sewage being treated in the tower. The 6 to 7.5 m (20 to 25 ft) of tower packing will normally produce a pressure drop of 15 to 40 mm (0.6 to 1.6 in) of water.

15.7 PERSISTENT ORGANICS REMOVAL

15.7.1 General

Persistent organic compounds are considered to be potentially toxic, are persistent in the environment and may biomagnify as they move up the food chain. These chemicals can be classified into groups including pesticides, pharmaceutical and personal care products (PPCPs) residuals, polyaromatic hydrocarbons (PAHs), brominated diphenyl ethers (BDEs), industrial chemicals and byproduct compounds.

Depending on the chemical characteristics, these compounds will remain in the liquid stream, be volatilized to the air or be sorbed to solids in the system. It should be noted, however, that as is the case for industrial treatment works, if the sewage treatment plant regularly receives these compounds, microorganisms within the biological process may be become acclimated and some biodegradation of the compounds may occur.

Currently, removal of persistent organics is not specifically designed for at municipal STPs in Ontario. The impact of any persistent organic substance should first be characterized prior to considering advanced removal technologies.

Persistent organics may be removed from the liquid stream through the use of activated carbon or by using membrane technology (*Section 15.4 - Membranes*).

Volatile persistent compounds may be stripped from the liquid phase to the gas phase through the aeration system in the case of aerobic treatment or by utilizing stripping equipment.

Chemical oxidation can be used to detoxify the persistent compounds. In this process, an oxidizing agent(s) is used to transform the chemical to either a less toxic compound or one that may be more biodegradable using a downstream biological process. Chemical oxidation technologies include the use of ozone, hydrogen peroxide or chlorine. Ultraviolet (UV) irradiation has been used in conjunction with chemical additives to accelerate the chemical oxidation process (i.e., advanced oxidation).

15.7.2 Activated Carbon Adsorption

In tertiary treatment, the role of activated carbon is to remove the relatively small quantities of refractory organics, as well as inorganic compounds such as nitrogen, sulphides and heavy metals, remaining in an otherwise well-treated effluent. Activated carbon may also be used to remove soluble organics following physical/chemical treatment.

The selection of an activated carbon adsorption process should be based on actual pilot test data and on the overall economy of the proposed treatment scheme, including the life-cycle cost of the carbon contact process and the cost of disposing or regenerating the spent carbon.

Carbon contact vessels, regeneration furnaces and other process equipment that may be vulnerable to severe climatic conditions should be enclosed in a building. The designer should consider using a modular design for future expansion of the carbon contact process. All structural shelters should have adequate heating and ventilation for the protection of personnel and equipment.

The required volume for a carbon contact vessel should be based upon expected organic and hydraulic loading conditions. In no case should the empty bed contact time be less than 15 minutes at all expected flow conditions.

Where the sewage effluent contains 20 mg/L suspended solids or more, further reduction of suspended solids by granular filtration should be considered prior to feeding activated carbon contactors. Where a gravity downflow contact vessel design is proposed, the maximum hydraulic loading should be 27 L/(m^2·s) (40 USgpm/ft^2) for all flow conditions.

The carbon contact process design should provide flexibility to operate the contact vessels in parallel or in two-stage series flow regimes. The contact vessel inlet and effluent collection should provide for uniform flow distribution throughout the bed volume.

Where a gravity downflow contact vessel is proposed, the vessel design should provide for 50 percent bed expansion during the backwash cycle. The backwash system should provide a range of flow from 8 to 17 L/(m^2·s) (12 to

25 USgpm/ft^2) of surface area. Additionally, supplemental surface wash at 12.8 to 25.7 L/(m^2·s) (18.9 to 37.8 USgpm/ft^2) should be provided to assist in backwashing the bed.

15.7.3 Oxidative Methods

Oxidative methods for toxic organics and colour destruction have expanded significantly due to technological advances in dealing with hazardous wastewater and *groundwater* remediation. Accepted techniques include chlorination and hydrogen peroxide addition as well as less conventional techniques such as UV/ozone or UV/hydrogen peroxide oxidation or advanced oxidation with UV and catalysts (e.g. titanium dioxide). The systems can offer an effective approach to toxic organic and color destruction while posing no sidestream treatment or disposal problems. Cyanides, phenols, aromatic organics and volatile organics have been effectively treated using such technologies. Oxidative methods can be combined with activated carbon to create a highly effective and reliable contaminant removal system. Optimization of oxidative methods may also involve pH adjustment, recycling of flows and extended detention depending on the contaminant to be removed.

15.7.4 Selective Ion Exchange

Ion exchange technology is typically utilized for the removal of heavy metals or ionically charged organics. Numerous natural and synthetic resins are commercially available, typically in bead or granular form. Ion-exchange resins can be classified as either cationic or anionic. The potential for resin fouling mandates an influent sewage stream low in suspended solids and organic content. Resins can often be regenerated on-site; however, provisions should be made to deal with the regeneration waste streams. Multiple units are essential for operation and during regeneration periods.

15.7.5 Reverse Osmosis and Membrane Filtration

Reverse osmosis (RO) represents a quaternary treatment technology, capable of providing an exceptional effluent quality, including the removal of dissolved organics. A high-quality influent sewage stream is essential to its operation. Extensive pretreatment is required to remove suspended solids, extreme pH values, oil and grease and membrane-destructive chemical constituents. Similar to ion-exchange methods, membranes can be cleaned on-site; however, they often generate significant waste streams (i.e., concentrate) both difficult to treat and requiring significant detention. Multiple-membrane systems are essential for operation, cleaning and during membrane replacement.

15.8 NATURAL SYSTEMS

Natural systems use natural vegetation to treat or polish sewage. Natural systems need to meet similar effluent quality criteria based on the assimilative capacity of the receiver as other treatments processes. This generally means limits for $CBOD_5$, TSS, total phosphorus and often TAN.

The design of any sewage treatment works should be based on the premise that failure of any single component should not prevent the works from meeting its effluent quality criteria and consequently should have a level of reliability and redundancy of its components commensurate with the stringency of the effluent quality criteria.

Due to the nature of constructed wetlands treatment technology, which is characterized by limited process control, the designer should consider utilization of wetland treatment for sites where it is a polishing component of the overall treatment process.

15.8.1 Constructed Wetlands

15.8.1.1 General

Constructed wetlands are land areas with water depths typically less than 0.6 m (24 in) that support the growth of emergent plants such as cattails, bulrushes, reeds and sedges. The vegetation provides surface for the attachment of bacterial films, aids in the filtration and adsorption of sewage constituents, transfers oxygen into the water column and controls the growth of algae by restricting the penetration of sunlight.

Although plant uptake is a consideration in nutrient removal it is only one of many active removal mechanisms in the wetland environment. Removal mechanisms have been classified as physical, chemical and biological and are operative in the water column, the humus and soil column beneath the growing plants and at the interface between the water and soil columns. Since most of the biological transformations take place on or near a surface to which bacteria are attached, the presence of vegetation and humus is very important.

15.8.1.2 Types

Sewage treatment systems using constructed wetlands have been categorized as either free water surface or subsurface flow types.

Free Water Surface (FWS) Wetlands - A FWS wetland system consists of basins or channels with a natural or constructed subsurface barrier to minimize seepage. Emergent vegetation is grown and sewage is treated as it flows through the vegetation and plant litter. FWS wetlands are typically long and narrow to minimize short-circuiting.

Subsurface Flow (SSF) Wetlands - A SSF wetland system consists of channels or basins that contain gravel or sand media which will support the growth of emergent vegetation. The bed of impermeable material is sloped typically between 0 and 2 percent. Sewage flows horizontally through the root zone of the wetland plants about 100 to 150 mm (4 to 6 in) below the gravel surface. Treated effluent is collected in an outlet channel or pipe.

15.8.1.3 Site Evaluation

Site characteristics that should be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard and climate.

Topography - Level to slightly sloping, uniform topography is preferred for wetland sites because free water systems are generally designed with level basins or channels and subsurface flow systems are normally designed and constructed with slopes of about 1 percent.

Soil - Sites with slowly permeable (i.e., <0.5 cm/h; <0.2 in/hr) surface soils or subsurface layers are most desirable for wetland systems because the objective is to treat the sewage in the water layer above the soil profile. Therefore, percolation losses through the soil profile should be minimized.

Flood Hazard - Wetland sites should be located outside of flood plains or protection from flooding should be provided.

Existing Land Use - Open space or agricultural lands, particularly those near existing natural wetlands, are preferred for wetland sites. Large areas are required due to shallow depths and long retention times of these systems.

Climate - Since the principal treatment processes are biological, treatment performance is temperature sensitive. Storage will be required where treatment objectives cannot be met due to seasonal low temperatures.

15.8.1.4 Pre-application Treatment

Constructed wetland sewage treatment systems should be designed with pretreatment. Since no permanent escape mechanism exists for phosphorus within the wetland, phosphorus reduction by chemical addition is needed before wetland treatment.

15.8.1.5 Vegetation Selection and Management

The plants most frequently used in constructed wetlands include cattails, reeds, rushes, bulrushes and sedges. All of these plants are ubiquitous in Ontario and can tolerate freezing conditions. The important characteristics of the plants related to design are the optimum depth of water for FWS systems and the depth of rhizome and root systems for SSF systems.

Harvesting of wetland vegetation is generally not required, especially for SSF systems. However, dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channeling of the flow. Removal of the plant biomass for the purpose of nutrient removal is generally not practical.

15.8.1.6 Design Parameters

Detention time is a key design parameter affecting the magnitude of CBOD5 removal. The range of typical detention time is 5 to 10 days (for SSF wetlands based on pore volume).

For FWS, water depths should range from 0.1 to 0.5 m (4 to 20 in). The design depth of SSF systems is controlled by the depth of penetration of the plant rhizomes and roots because the plants supply oxygen to the water through the root/rhizome system. The media depth may range from 0.3 to 0.8 m (12 to 30 in).

The aspect ratio for FWS wetlands is important to the performance; length to width (L/W)ratios of 4:1 to 6:1 are needed to achieve good performances and avoid short-circuiting of sewage through the wetland. Even for large systems, an aspect ratio should never be smaller than 2:1.

For SSF wetlands the bed width is determined by the hydraulic flow rate. The length of the bed is determined by the needed detention time for pollutant removal. Therefore SSF wetlands may have aspect ratios less than or greater than 1:1 depending on the treatment goal.

Table 15.1 summarizes the hydraulic, BOD_5 and TSS loading rates for organics removal in both FWS and SSF systems.

Table 15-1 - Loading Rates for Constructed Wetlands

Wetland Type	Hydraulic Loading Rate	Maximum BOD_5 Loading Rate	Maximum TSS Loading Rate at Inlet
Free Water Surface System	150 - 500 $m^3/(ha \cdot d)$ [16,040-53,450 US gal/(ac·d)]	65 kg/(ha·d) [58 lb/(ac·d)]	Not Applicable
Subsurface Flow System	Not Applicable	65 kg/(ha·d) [58 lb/(ac·d)]	0.08 $kg/(m^2 \cdot d)$ [0.02 $lb/(ft^2 \cdot d)$]

For nutrient removal, the following should be considered for each type of wetland:

- **Free Water Surface Wetlands** - Detention times for ammonia nitrogen removal need to be longer than the 5 to 10 days required for organics removal. For ammonia or total nitrogen removal, both minimum temperature and detention time are important. Detention times for significant nitrogen removal should be 8 to 14 days or more. Nitrification will be reduced when water temperatures fall below 10 ºC (50 ºF) and should not be expected when water temperatures fall below 4 ºC (39 ºF). Plant uptake of phosphorus is rapid and following plant death, phosphorus may be quickly recycled to the water column or deposited in the sediments. The only major sink for phosphorus in most wetlands is in the soil. Significant phosphorus removal requires long detention times (15 to 20 days) and low phosphorus loading rates [(< 0.3 kg/(ha·d)][< 0.3 lb/(ac·d)]; and

- **Subsurface Flow Wetlands** - Both detention time and oxygen transfer can limit nitrification and subsequent nitrogen removal in SSF wetlands. Oxygen transfer is critical to nitrification in SSF wetlands. Plant roots can transfer a portion of this demand for oxygen in the subsurface; however, direct oxygen transfer from the atmosphere may be required to achieve effective nitrification. The detention time and temperature limits for FWS wetlands apply to SSF wetlands.

15.8.1.7 Vector Control

FWS systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of mosquito fish and sparrows plus application of chemical control agents as necessary should be incorporated in the design. Thinning of vegetation may also be necessary to eliminate pockets of water that are inaccessible to fish.

15.8.2 Aquatic Plant Treatment Systems

Aquatic plant treatment systems consist of one or more shallow ponds in which one or more species of water tolerant vascular plants such as water hyacinths or duckweed are grown. The shallower depths and the presence of aquatic macrophytes in the place of algae are the major differences between aquatic treatment systems and stabilization ponds. In these systems, sewage is treated principally by bacterial metabolism and physical sedimentation. The aquatic plants themselves do not provide significant treatment of the sewage. Their function is to provide components of the aquatic environment that improve the sewage treatment capability and/or reliability of that environment.

The principal floating aquatic plants used in aquatic treatment systems are water hyacinth, duckweed and pennywort.

The minimum level of pre-application treatment should be primary treatment, short detention time aerated ponds or the equivalent. Treatment beyond primary levels depends on the effluent requirements. Use of oxidation ponds or lagoons in which high concentrations of algae are generated should be avoided prior to aquatic treatment. When there are effluent limitations on phosphorus, it should be removed in the pre-application treatment step because phosphorus removal in aquatic treatment systems is minimal.

The water hyacinth systems that are currently used to treat sewage are located in the warm temperature climates. The optimum water temperature for water hyacinth growth is 21 to 30 ºC (70 to 86 ºF). An air temperature of -3 ºC (27ºF) for 12 hours will destroy the leaves and exposure to -5 ºC (23 ºF) for 48 hours will kill the plants. If a water hyacinth system were to be used in a colder climate, it would be necessary to house the system in a greenhouse and maintain the temperature in the optimum range. Duckweed is more cold tolerant than water hyacinths and can be grown practically at temperatures as low as 7 ºC (45 ºF).

Overall, these systems are not generally applicable for use in Ontario other than in a greenhouse environment and are not discussed in any additional details.

15.9 LAND APPLICATION OF TREATED EFFLUENT

Land application of treated effluent is a method of disposing of the final effluent without direct discharge to *surface waters*. The minimum level of treatment required for direct surface discharges in Ontario is secondary treatment. Land application is a disposal alternative that can be used when there is insufficient assimilative capacity in nearby watercourses or where downstream water uses will preclude direct effluent discharges. Other disposal methods, such as subsurface disposal via tile fields or pipeline conveyance to more acceptable receiving streams, should also be considered as alternatives to land application of effluent.

Land application of treated effluent takes advantage of the soil and vegetation capacities to renovate effluent by the combined processes of filtration, adsorption, chemical precipitation, ion exchange, biochemical transformation and/or biological absorption. There are a number of land application techniques, including spray irrigation, rapid infiltration basins, ridge and furrow systems and overland runoff systems. Spray irrigation has been the most widely used method of land application in Ontario.

For successful operation, an effluent irrigation system will require:

- Suitable soils;
- Suitable topography and hydrological conditions;
- Adequate site area at reasonable cost;
- Suitable site isolation from conflicting land uses;
- Suitable climate;
- Effective site preparation;
- Proper crop selection;
- Good management;
- Adequate sewage treatment prior to irrigation; and
- Adequate effluent holding capacity for non-irrigation periods.

15.9.1 Soils

A soils report should be prepared before the design of the land application system. This report should not only demonstrate the suitability of the soils for sewage lagoons construction which is normally required with land application systems, but also the acceptability of the infiltration capacity and permeability of the soil to accommodate the proposed final effluent application rates.

The infiltration capacity refers to the rate at which water can enter the soil. If the application rate exceeds this capacity, surface ponding, runoff and erosion will occur, leading to deterioration in soil structure and a further decrease in infiltration capacity. This effect is particularly important with soils containing silt and clay.

Permeability refers to the ability of the soil to allow water to move through the soil. Permeability will generally vary with depth, with the surface soils generally being more permeable than subsoil. Soils testing should, therefore, determine the limiting permeability of the soils at the proposed application site.

Other factors that the soils report should establish are the soil type and drainage characteristics, the soil strata and the expected depth to the water table during the irrigation season.

15.9.2 Topographical and Hydrological Conditions

A contour plan, showing contours not exceeding 0.5 m (2 ft) intervals, should be prepared for the treatment and irrigation areas. The present and future directions of surface drainage and groundwater movement from the spray irrigation site should be determined and shown on the topographical plan. Irrigation in areas close to watercourses may have to be excluded depending upon the runoff coefficient for the site and the distance from the watercourse. Designers of sewage effluent irrigation systems need to demonstrate that the environmental impacts that will be caused by the system are compatible with existing and potential land use. The *ministry* Guideline B-7, *Incorporation of the Reasonable Use Concept into Groundwater Management* provides the framework for determining acceptable off-property impacts on groundwater resources. For more detailed information on application of the Guideline B-7, the designer should refer to *Section 22.5 – Assessment of Impact on Water Resources.*

Spray irrigation sites should be as flat as possible to facilitate agricultural activities and to minimize runoff. Slopes on cultivated fields should be limited to 4 percent. On grassland, slopes of 3 percent may be acceptable. Steeper slopes on forested land may be acceptable, depending upon the period of spray irrigation. Depressions or ruts within spray areas should be filled or avoided to prevent stagnation or channelization of the effluent.

Consideration should be given to the need for emergency discharges of effluent from the treatment facilities. The method by which such discharges could take place and the route such discharges would follow to a watercourse should be defined.

The depth to the water table in the irrigation area should be at least 2 m (6 ft), unless the site is underdrained, in which case a drain depth of 1 m (3 ft) is satisfactory.

Soil permeability in the moderate to rapid class (10^{-4} to 10^{-2} cm/s) ($40x10^{-6}$ to $40x10^{-4}$ in/s) is generally considered ideal for spray irrigation systems. The spray area should be designed in such a way that surface runoff does not enter or leave the spray area.

15.9.3 Site Area Requirements

Various factors influence the size requirements for the irrigation area, including the following:

- Length of irrigation season;
- Volume of effluent to be applied; and
- Acceptable average application rate over the irrigation season.

For infiltration-percolation systems, the frost-free period is the recommended limit for the length of the irrigation season when the land is not underdrained. When the land is underdrained, the mean annual growing season is the recommended limit for the length of the irrigation season for infiltration-percolation and overland runoff systems. For irrigation systems relying primarily upon evapotranspiration (minimum infiltration and runoff), the limit of the irrigation season will be the frost-free period.

In Ontario, the mean frost-free period ranges from a high of 172 days in the climatic Region of Leamington to 75 days in the climatic Region of Patricia. For the same regions, the growing seasons are 221 and 131 days, respectively.

The amount of final effluent which may be applied by spray irrigation over a season will depend upon the infiltration/permeability of the soil and the crop water deficit. The crop water deficit is the sum of the potential evapotranspiration and the soil moisture holding capacity minus the May to September precipitation. The crop water deficit is very small and usually amounts to only a few centimetres of liquid per year.

Regardless of the length of the frost-free period and the calculated average seasonal effluent application rate, the spray irrigation site area cannot be based upon a spray season in excess of 100 days nor upon an average effluent application rate in excess of 55,000 L/(ha·d) [5,900 US gal/(ac·d)].

15.9.4 Site Buffer Zones

From the outside limits of sewage lagoons to dwellings, an isolation distance of at least 100 m (330 ft) should be provided. For larger lagoon facilities, distances up to 400 m (1300 ft) may be required to minimize odour problems.

In the absence of detailed assessments, the distance from spray nozzles to the property limit should be 150 m (490 ft). Spraying is possible at closer distances from the property limit provided that low pressure, low angle, closely spaced sprinklers are used to minimize the formation of aerosols. In addition, the risk associated with aerosols can be minimized by providing a fence or tree screen around the site perimeter and by terminating spraying operations when wind speeds exceed that of a gentle breeze of 15 km/hr (9.3

mi/hr). Effluent disinfection should also be considered in addition to the above measures.

Lagoon and irrigation areas should be enclosed with suitable fencing to exclude livestock and to discourage trespassing. Vehicle access gates should be provided where necessary to accommodate maintenance and supply vehicles and agricultural equipment. All access gates should be locked. The perimeter fences and gates should be provided with appropriate signs designating the nature of the facility and prohibiting trespassing.

15.9.5 Pilot Testing

On-site pilot testing is recommended to determine the feasibility of land application of treated effluent and to provide design data on application rates and quantities.

15.9.6 Treatment Requirements

Treated effluent cannot be irrigated on crops used for direct human consumption. Land which has been previously irrigated with secondary effluent, or equivalent, can be used for such crops, provided that a period of at least 6 months has elapsed since the last effluent application.

With crops used for animal consumption, land application of sewage treatment lagoon effluent or normally disinfected (chlorination at 0.5 mg/L residual and 30 minute contact time) secondary effluent from other treatment processes may be used.

For dairy cattle pastures, the sewage should have received the equivalent of secondary treatment plus disinfection to the bacteriological criteria for swimming and bathing use of water (geometric mean densities of less than 100 *E. coli* per 100 mL). Treatment provided by a facultative lagoon is designed to the criteria outlined in *Section 12.3.1.1 - Facultative Lagoons* for seasonal discharges. At least 30 days retention time since the last addition of raw sewage prior to spraying is considered equivalent to secondary treatment and may achieve the above mentioned bacteriological criteria without disinfection being required.

For pasture, silage, haylage, orchards, and other food crops, the effluent should be normally disinfected (chlorination to 0.5 mg/L residual and 30 minutes contact time). For orchards, non-spray application methods should be used, (e.g. ridge and furrow or gated pipe). In all cases, the crop should be allowed to dry before harvesting or pasturing.

In all of the above cases, if the land is not to be used for at least one-half year after spraying, disinfection will not be necessary.

With recreational lands such as golf courses, the treatment requirement is secondary biological activated sludge treatment or equivalent, with the resulting effluent being discharged to the first of two ponds connected in series, each with a retention period of not less than 30 days. The effluent to be sprayed should be disinfected (chlorination at 0.5 mg/L residual and 30 minute contact time or equivalent).

15.9.7 Crop Selection

On arable land, perennial grasses (Brome orchard, Reed Canary and Timothy) are most suitable for spray irrigation disposal sites as they have fibrous root systems, are sod forming, which aids in erosion control, provides for high infiltration rates, have a long period of growth and have a high uptake of nutrients. The order of preference for use of these grasses on disposal sites is provided in Table 15-2.

Table 15- 2 - Preference for Grasses Used at disposal Sites

Order	Name	Comments
1	Reed Canary	Tolerates excessive moisture and is highly productive for long term hay or pasture on poorly drained soils or areas subject to prolonged periods of flooding; less palatable than other grasses; more acceptable to livestock when stored as silage or haylage rather than dry hay.
2	Timothy	Well adapted to heavier soil types and variably drained soils.
3	Brome	Highest digestibility of grasses when cut at the "heads emerged" stage; superior for early pasture; good growth in fall.
4	Orchard	Requires well-drained sites to avoid winter kill; grows back immediately after cutting or grazing.

Forests and brushland should also be considered for effluent disposal as the land value is relatively low compared to cultivated areas.

15.9.8 Moisture Requirements and Effluent Application Amounts

The moisture requirements for perennial grasses grown in various soil types are shown in the Table 15-3. The application amount should be applied over a one-day period and should not be repeated until the number of days specified in the period between irrigation applications has passed. The cycle is then repeated throughout the irrigation season.

The application amount is the quantity which should be needed to maintain the soil at "Field Capacity". Any combination of rainfall or effluent quantities above this amount will percolate through the soil to the water table. If the sum of the effluent application and rainfall is equal to or less than the "application amount", maximum plant utilization of the effluent nutrients and minimum infiltration to the groundwater should occur.

The effluent application amounts for infiltration and evapotranspiration spray irrigation systems should be as follows:

- The hourly effluent application rate should be less than the surface infiltration rate measured in cm/hr (in/hr).
- The daily effluent application rate plus the weekly rainfall on an area of the spray field that is irrigated one day per week measured in cm/d (in/hr) should be less than the permeability of the most impermeable soil sub-horizon measured in cm/wk (in/wk).

Table 15 -3 - Moisture Requirements for Perennial Grasses [1]

Soil	Application Amount (cm) [in]	Period Between Irrigation Applications (days)	Recommended Application Rate (cm/hr) [in/hr]
Well Drained Sands	3.3 [1.3]	5	0.6-1.9 [0.24-0.75]
Loamy Sands	4.3 [1.7]	6	0.6-1.3 [0.24-0.51]
Light Coloured Loams and Sandy Loams and Good Drainage	5.1 [2.0]	7	0.6-1.3 [0.24-0.51]
Dark Coloured Loams and Sandy Loams with Fair to Poor Drainage	6.9 [2.7]	10	0.6-1.3 [0.24-0.51]
Clay Loams	6.1 [2.4]	9	0.4-1.0 [0.16-0.39]

Note:

1. This information is from *Irrigation Practices for Ontario*, OMAF AGDEX 560/753.

15.9.9 Spray Irrigation System Design

Spray irrigation areas should be divided into sections such that spraying can be carried out on a rotation basis with the most effective use being made of the spray area and equipment.

Irrigation piping in the spray areas should allow for flexible operation, including selection of spray areas and isolation of piping sections. Although stationary piping systems may be used for small systems, traveling sprinklers may also be considered provided the topography is suitable. Stationary piping systems should be designed to permit drainage to prevent freezing damage. Valves, sprinkler heads and pipelines should be colour coded and designated as carrying treated sewage to prevent cross connections and improper use.

Sprinklers should be provided in such a pattern that full field coverage is achieved. Some overlapping of the spray patterns will be necessary to ensure total coverage. Non-irrigated areas would result in excessive weed growths with lower crop values in the drier areas.

A flow meter should be provided to permit measurement of the application rates and amounts. A pressure gauge should also be provided to monitor line and sprinkler head losses.

15.9.10 Site Control

The designer of irrigation systems should be able to demonstrate that the irrigation lands will be available when needed to dispose of effluent. This will normally mean that the lands should be owned by the proponent.

Non-ownership of the irrigation lands may be considered provided that the lands are leased over a long enough term and with renewal clauses to ensure that alternate disposal options could be developed, if found necessary. The terms of the lease should also grant the owner of the sewage treatment system the right to irrigate even if such action may destroy or damage the crops being grown. This latter provision will be necessary to ensure that the satisfactory disposal of effluent will take priority over any cropping activities when and if, found necessary. The lease should include terms to compensate the landowner for crop damage or loss in such eventualities.

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CHAPTER 16

SLUDGE STABILIZATION

CHAPTER 16

SLUDGE STABILIZATION

This chapter describes treatment of waste solids (sludge and biosolids) from sewage treatment plants, including information on sludge quantity and concentration, biological anaerobic and aerobic stabilization and storage. Alkaline stabilization, thermal drying, solar drying, composting and incineration, are also discussed. Enhanced biosolids processing is used to provide either increased stabilization, especially for pathogen reduction, for reuse or land application, or maximum volume reduction by incineration and diversion away from land application.

The treatment of sewage results in the production of solids commonly referred to as sludge. Sludge that has undergone dewatering or treatment is generally referred to as treated sludge. When the treated sludge is suitable for land application, it is referred to as biosolids.

A summary of design loading and sludge quantity data for conventional sludge stabilization processes is provided in Appendix V, which should be used in conjunction with the details in this chapter.

16.1 GENERAL

Sludge stabilization is generally achieved by digestion and these guidelines deal with two types of sludge digestion processes that are commonly used in Ontario - anaerobic and aerobic. Alkaline stabilization is discussed, although it is not in broad use in Ontario at this time. However, proprietary alkaline stabilization systems are available and used in Ontario.

Anaerobic mesophilic digestion is the most commonly used process for the digestion of primary and mixtures of primary and waste secondary treatment sludges (e.g. waste activated sludge), particularly at larger plants. In anaerobic digestion systems, methane and other hazardous gases are created. Access to any enclosed spaces and ventilation of these spaces should follow all required safety codes. It is essential to review the Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation*, CAN/CGA-B105-M93, 1993 and the National Fire Protection Association's (2003) *Fire Protection in Wastewater Treatment and Collection Facilities*, US NFPA 820. The latest editions of these codes should be applied in designing digestion systems, as they provide comprehensive details concerning gas handling and safety.

Aerobic digestion has normally been used in Ontario for the stabilization of waste activated sludges from sewage treatment plants which do not have primary sedimentation tanks. Aerobic digestion is most common at smaller plants. Although there are some aerobic systems treating mixtures of raw, primary and waste activated sludges, due to the higher oxygen requirements and associated higher energy costs, it is recommended that the aerobic digestion process generally not be used for such sludges in new plants.

Autothermal aerobic digestion systems (ATAD) that operate at thermophilic condition have been constructed in Ontario and elsewhere.

Facilities for processing sludge should be provided at all sewage treatment plants, unless facilities are available and can be used at another facility, in which case only storage may be needed. Handling equipment should be capable of processing sludge to a form suitable for ultimate disposal unless provisions acceptable to the *ministry* are made for processing the sludge at an alternate location.

Designers are cautioned to give thorough consideration to what type of digestion will best suit a particular sewage treatment plant (STP) and what type of overall system, including plant type and digestion type will produce the desired results economically. A process control narrative (including level of automation) should be developed and agreed on by all parties (owner, operator and consultant) during the pre-design stage.

In certain plants, separate processing of the waste activated sludge (WAS) in aerobic digesters and primary sludge in anaerobic digesters may be economically justifiable, but this is not common in Ontario or elsewhere.

Sludge handling, digestion and disposal represent a large portion of the overall sewage treatment operation and may impact other unit processes at the STP. No one unit operation can be considered in isolation from the other plant components. If sludge unit processes not described in this chapter are being considered or are necessary to meet provincial or federal sludge disposal requirements, details of the process need to be evaluated and included in the design brief.

The *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management Act* should be referred to for criteria for land application of biosolids, e.g. seasonal storage requirement. For STPs which are not phased in under the *Nutrient Management Act*, requirements are set out in the Certificate of Approval (C of A), based on the MOE and the Ministry of Agriculture, Food and Rural Affairs' *Guidelines for the Utilization of Biosolids and Other Wastes on Agricultural Land, 1996*.

16.1.1 Sludge Quantities and Characteristics

An overall STP mass balance should be provided to account for sludge production from each treatment unit process, including the liquid and solids trains and the influence of recycle streams. Steady state or dynamic simulation models can be used to assist the designer with accurately estimating a plant-wide solids and nutrients mass balance. At minimum the total and volatile solids loading rate to the sludge processing units should be estimated.

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester or other sludge treatment process design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. A good check of the data is to conduct a sludge accountability analysis based on the primary or raw, biological and chemical sludge expected for a plant. The quantity of primary or raw sludge generated can be based on field data from a similar or actual installation taking into account removal rates. The biological sludge production can be calculated using the current BOD_5 loading and typical unit sludge values. Unit biological sludge values vary from 0.70 kg TSS/kg BOD_5 (0.70 lb TSS/lb BOD_5) removed for activated sludge plants with primary clarifiers to 0.85 kg TSS/kg BOD_5 for activated sludge plants without primary clarifiers. The amount of chemical sludge is calculated from the stoichiometry. Details for this method are available in ministry publication *The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants* (1996).

Before sludge data are used for design, they should be assessed for their accuracy. When reliable data are not available, the sludge generation rates and characteristics given in Table 16-1 may be used to initiate the estimation of sludge quantity. However, a comprehensive plant mass balance is recommended and should be provided.

The designer should refer to *Chapter 17 - Sludge Thickening and Dewatering* for methods for sludge thickening and dewatering that may be applicable to digestion practices. Mechanical thickening prior to digestion can reduce digester hydraulic loading and thus increase digester hydraulic and solids retention time and reduce heating requirements. Mechanical thickening can also reduce digester supernatant return loads to the liquid train of the STP.

In addition to sludge thickening prior to digestion, wherever possible, separate removal of grit, oils and greases and other debris such as rags is recommended to prevent entry into the digestion process.

Pretreatment of WAS or thickened WAS by a process that induces cell lysis could be considered, to increase VS destruction and gas generation in the anaerobic digestion system.

16.1.2 Process Selection

The selection of sludge handling and treatment processes should be based upon at least the following considerations:

- End use of sludge or biosolids;
- Plant size and economy-of-scale;
- Sludge characteristics (e.g. quality and proportion of various types of solids);
- Odour control;
- Local land use;
- System energy requirements;

Table 16-1 - Typical Sludge Generation Rates and Characteristics[1]

Unit Process	Liquid Sludge[2] L/m³ (USgal/1000ft³)	Solids Concentration Range (%)	Solids Concentration Average (%)	Volatile Solids (%)	Dry Solids g/m³ (lb/1000 ft³)[3]	Dry Solids g/(cap·d)
PRIMARY SEDIMENTATION WITH ANAEROBIC DIGESTION						
Undigested (No P Removal)	2.0 (15)	3.5 – 6	5.0	65	120 (7.5)	55
Undigested (With P Removal)	3.2 (24)	3.5 – 7	4.5	65	170 (10.6)	77
Digested (No P Removal)	1.1 (8.2)	5 – 8	6.0	50	75 (4.7)	34
Digested (With P Removal)	1.6 (12)	5 – 8	6.0	50	110 (6.9)	50
PRIMARY SEDIMENTATION AND CONVENTIONAL ACTIVATED SLUDGE WITH ANAEROBIC DIGESTION[4]						
Undigested (No P Removal)	4.0 (30)	2 – 7	4.5	65	160 (10.0)	82
Undigested (With P Removal)	5.0 (37)	2 - 6.5	4.0	60	220 (13.7)	100
Digested (No P Removal)	2.0 (15)	2 – 6	5.0	50	115 (7.2)	52
Digested (With P Removal)	3.5 (26)	2 – 6	4.0	45	150 (9.4)	68
CONTACT STABILIZATION AND HIGH RATE ACTIVATED SLUDGE WITH AEROBIC DIGESTION[5]						
Undigested (No P Removal)	15.5 (116)	0.4 - 2.8	1.1	70	170 (10.6)	77
Undigested (With P Removal)	19.1 (143)	0.4 - 2.8	1.1	60	210 (13.1)	95
Digested (No P Removal)	6.1 (46)	1 – 3	1.9	70	115 (7.2)	52
Digested (With P Removal)	8.1 (61)	1 – 3	1.9	60	155 (9.7)	70
EXTENDED AERATION WITH AERATED SLUDGE HOLDING TANK						
Undigested Waste Activated (No P Removal)	10.0 (75)	0.4 - 1.9	0.9	70	90 (5.6)	41
Undigested Waste Activated (With P Removal)	13.3 (99)	0.4 - 1.9	0.9	60	120 (7.5)	55
Sludge Holding Tank (No P Removal)	4.0 (30)	0.4 - 4.5	2.0	70	80 (5.0)	36
Sludge Holding Tank (With P Removal)	5.5 (41)	0.4 - 4.5	2.0	60	110 (6.9)	50

Notes:

1. The above values are based on raw sewage with BOD_5 = 150-200 mg/L, Soluble BOD_5 = 50% of BOD_5, TSS = 150-200 mg/L, TP = 6-8 mg/L, TKN = 30-40 mg/L, TAN = 20-25 mg/L.
2. (L/m^3) denotes litres of liquid sludge per cubic metre of treated sewage.
3. (g/m^3) denotes grams of dry solids per cubic metre of treated sewage.
4. Typical primary sludge concentration will tend in the low end of the range listed with co-thickening of waste activated sludge.
5. Waste activated sludge mass rate is approximated by the aeration tank volume (V) and the target SRT. The volumetric waste activated sludge rate (W) is governed by the SRT and the ratio of the MLSS concentration and the recycle concentration (X_R) [$(W=V \cdot X_m/X_R \cdot SRT)$].

- Cost effectiveness of sludge thickening and dewatering;
- Equipment complexity and staffing requirements;
- Adverse effects of heavy metals and other sludge components upon the unit processes;
- Sludge digestion or stabilization requirements, including appropriate pathogen and vector attraction reduction;
- Sidestream or return flow treatment requirements (e.g. supernatant from digestion or sludge storage facilities, dewatering unit filtrate, wet air oxidation return flows);
- Sludge storage requirements;
- Methods of ultimate disposal or utilization; and
- Back-up techniques of sludge handling and disposal.

16.2 ANAEROBIC SLUDGE DIGESTION

16.2.1 General

Anaerobic digestion may be provided using mesophilic (35°C or 95°F) or thermophilic (55°C or 131°F) temperatures. Active digestion results in volatile solids reduction and gas production. The basis of design should be supported by sewage analyses to determine the presence of undesirable materials, such as high concentrations of sulphates and inhibitory concentrations of heavy metals.

16.2.2 Mesophilic Anaerobic Digestion

Two-stage mesophilic anaerobic digestion has typically been practiced at larger STPs in Ontario. This arrangement with primary and secondary digesters is considered to be high-rate digestion, consisting of a heated and mixed primary digester and an unheated and unmixed secondary digester. Multiple units in each stage may be required, depending on plant size.

Two stages with a minimum of one digester in each stage should be provided in all plants. Facilities for treated sludge or biosolids storage and supernatant separation in an additional unit may be required, depending on raw sludge concentration and disposal methods for biosolids and supernatant. In the case where only one digester is provided in each stage, piping should be designed to allow either stage to receive raw sludge with measures to allow for mixing and heating. The number of digesters in each stage of larger plants should be dictated by economics.

16.2.2.1 Process Variables

Anaerobic digester geometry has largely been based on low profile cylindrical vessels. Sidewall depth-to-diameter ratio typically ranges from 0.3 to 0.7. Shallower tanks are not conducive to good mixing. A freeboard allowance for scum and foam should be provided. In addition, bottom sediments can reduce active digestion volume.

If process design provides for supernatant withdrawal, the proportion of depth to diameter should be designed to allow for the formation of a reasonable depth of supernatant liquor. A minimum sidewater depth of 6.1 m (20 ft) is recommended.

New geometrical high rate anaerobic digester design includes “egg-shaped” digesters. These digesters do not normally provide for internal gas storage, but are claimed to provide better control of scum and bottom deposit formation and better active zone mixing. Higher ratios of height to diameter in cylindrical digesters may provide similar benefits; however, heating losses will need to be verified in design.

In addition to two-stage digestion, two-phase digestion is an innovative alternative that separates the acid forming and methane forming reactions of the primary digestion process in separate vessels. This has not been practiced to-date in Ontario. Other innovative staging options exist including temperature-phased anaerobic or mixed aerobic/anaerobic digestion process.

Provision for digester clean-out should be provided in design together with redundancy of tankage or convertibility of secondary digesters to primary digestion. Fine grit and other debris have a tendency to accumulate in digesters over time, reducing the effective digestion volume. To facilitate emptying, cleaning and maintenance, the features outlined in the subsection below are desirable for cylindrical digestion tanks.

16.2.2.2 Design Considerations

The following design considerations should be assessed:

- The tank bottom should slope to drain toward the withdrawal pipe. For tanks equipped with a suction mechanism for sludge withdrawal, a bottom slope not less than 1 to 12 is recommended. Where the sludge is to be removed by gravity alone, a 1 to 4 slope is recommended;
- The Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation*, CAN/CGA-B105-M93, 1993 specifies access manhole numbers and dimensions and the most up-to-date version of this code should be reviewed for details. There should be stairways and catwalks to reach the access manholes;
- A separate side wall manhole should be provided that is large enough to permit the use of mechanical equipment to remove grit and sand. The side wall access manhole should be low enough to facilitate heavy equipment handling and may be buried in the earthen bank insulation;
- Access to any tanks or enclosed spaces should be provided in accordance with the *Confined Spaces Regulation* (O. Reg. 632/05) made under the *Occupational Health and Safety Act*). These enclosed spaces may contain gases that result in respiratory failure unless proper approved methods for entry are followed;

- Non-sparking tools, rubber-soled shoes, safety harness, gas detectors for flammable and toxic gases and at least two self-contained breathing units should be provided for emergency use;
- Multiple sludge inlets and draw-offs and, where used, multiple recirculation suction and discharge points to facilitate flexible operation and effective mixing of the digester contents should be provided;
- Maximum flexibility should be provided in terms of sludge transfer from primary and secondary sewage treatment units to the digesters, between primary and secondary digesters and from the digesters to subsequent treated sludge or biosolids handling operations. The minimum diameter of sludge pipes should be 150 mm (NPS-6). Provision should be made for flushing and cleaning of sludge piping. Sampling points should be provided on all sludge lines. Main sludge lines should be from the bottom of the primary digester to the midpoint of the secondary digester, if pumped. Additional transfer lines should be from intermediate points in the primary digester (these can be dual purpose supernatant and sludge withdrawal lines);
- One inlet should discharge above the liquid level and be located at approximately the center of the tank to assist in scum breakup. The second inlet should be opposite to the suction line at approximately the 2/3 diameter point across the digester;
- Raw sludge inlet points should be located to minimize short-circuiting to the digested sludge or supernatant draw-offs;
- Treated sludge withdrawal to disposal should be from the bottom of the tank. The bottom withdrawal pipe should be interconnected with the necessary valving to the recirculation piping to increase operational flexibility in mixing the tank contents; and
- An unvalved vented overflow should be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This emergency overflow should be piped to an appropriate point and at an appropriate rate to the STP liquid train or sidestream treatment facilities to minimize the impact on process units.

16.2.2.3 Tank Design Capacity

The total digestion tank *capacity* should be determined by rational calculations based upon such factors as: sewage characterization; sewage treatment processes; volume of sludge added, percent solids and character; the temperature to be maintained in the digesters; the degree or extent of mixing to be obtained; degree of volatile solids reduction required; solids retention time at peak loadings; method of sludge disposal and size of the installation with appropriate allowances for gas, scum, supernatant and digested sludge storage.

The nominal minimum hydraulic retention time (HRT) in the primary digester should be at least 15 days [the theoretical solids retention time (SRT) requirement of slowest methane producers is approximately 10 days]. Secondary digesters of two-stage series digestion systems, which are used for digested sludge storage and concentration, should not be credited in the calculations for volumes required for sludge digestion.

Calculations should be prepared to justify the basis of design. The minimum digestion tank capacity outlined below is required. Such requirements assume that the raw sludge is derived from ordinary domestic sewage, a digestion temperature is to be maintained at 35°C (95°F), 40 to 50 percent volatile matter in the digested sludge, and that the digested sludge will be removed frequently from the process.

The secondary digester should be sized to permit solids settling for decanting and solids thickening operations and, in conjunction with off-site facilities, to provide necessary digested sludge storage. The storage volume should be based on the highest and lowest level in the digester without losing the gas seal. The necessary total storage time will depend on the means of ultimate sludge disposal, with the greatest time required with soil conditioning operations and with less storage required with landfilling or incineration as ultimate disposal methods.

16.2.3 Digested Sludge Storage

Onsite or offsite storage in sludge lagoons, sludge storage tanks, or other facilities may be used. If high rate primary digesters are used and efficient thickening within the secondary digester is required, the secondary digester should be conservatively sized to allow adequate solids separation (secondary to primary volume ratios of 2:1 to 4:1 are recommended). The secondary digester provides relatively little storage capacity, particularly if a land application program is practiced.

The designer should refer to *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management Act* for minimum biosolids storage requirement for a biosolids land application program.

16.2.4 Digester Design Loadings

The volatile solids loading rates to mesophilic anaerobic digesters should be designed as outlined below:

Completely Mixed Systems – High Rate Digestion

For digestion systems providing for intimate and effective mixing of the digester contents, the system may be loaded up to 1.6 kg/($m^3 \cdot d$) of volatile solids (100 pounds per 1000 cubic ft of volume per day) in the active primary digestion units. Higher digester loading rates have been proposed for well-mixed systems.

Moderately Mixed Systems – Low Rate Digestion

For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 0.65 kg/(m^3·d) of volatile solids (40 pounds per 1000 cubic feet of volume per day) in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided.

16.2.5 Digester Mixing

Primary anaerobic digesters can potentially experience severe losses of active mixing volume to dead space and short-circuiting of raw sludge past the mixing zone and appearing in the digester outlet. Tracer tests are recommended to determine the state of digester mixing as temperature or solids profiles do not alone provide evidence of mixing degree.

There are three main mixing techniques:

- Gas - confined and unconfined;
- Mechanical; and
- Pump and *in situ* mixers.

Gas mixing systems recirculate compressed digester gas in either unconfined or confined mixing. Both create upward mixing actions.

Mechanical mixing uses axial flow propellers with roof- or external-mounted draft tubes. The roof mounted draft tubes limit the digester size to less than 24 m (80 ft) in diameter; whereas, the external mounted tubes can accommodate diameters of 24 m (80 ft) and greater. Mechanical mixers using vertical mixing action can also be considered.

Pump mixing uses axial flow patterns, and screw-type centrifugal or chopper-type features. This draws sludge from the bottom and pumps it back into the top. *In situ* mixers include the internal plunger cam design.

Where sludge recirculation pumps are used for mixing they should meet design recommendations contained in *Section 16.7 - Sludge Pumps and Piping*.

Typical power requirement for primary stage mixing ranges between 5 to 8 W/m^3 [0.2 to 0.3 hp/(1000 ft^3)] for compressed gas mixing and 6.6 W/m^3 [0.25 hp/(1000 ft^3)] for mechanical mixing.

The designer's calculations of the actual power requirements should be based on tank size, sludge rheology, types of mixers, mixer performance and mixing energy or shear rate required.

One concern with mixing is the formation of foam and grease on the digester surface. Means of foam/grease removal or suppression should be defined. Digesters treating thickened WAS may have significantly different mixing requirements than digesters not treating such a feed stream.

16.2.5.1 Digestion Tank Heating

If digestion tanks are constructed above grade level they need to be suitably insulated to minimize heat loss. Maximum utilization of earthen bank insulation should be used.

Sludge may be heated by circulating the sludge through external heaters or by units located inside the digestion tank. The system may be designed to provide for the preheating of feed sludge before introduction into the digesters. Grinder-type pumps are recommended to avoid heat exchanger clogging. Provisions should be made in the layout of the piping and valving to facilitate heat exchanger tube removal and cleaning of the lines. Heat exchanger sludge piping should be sized for peak heat transfer requirements. Heat exchangers should have a heating capacity of 130 percent of the calculated peak heating requirement to account for the occurrence of sludge tube fouling.

The use of hot water heating coils affixed to the walls of the digester, or other types of internal heating equipment that require emptying the digester contents for repair, are not acceptable.

Other systems and devices have been developed to provide both mixing and heating of anaerobic digester contents. These systems should be reviewed on their own merits. Operating data detailing their reliability and operation and maintenance characteristics should be reviewed. Process calculations should be documented to demonstrate that sufficient heating and mixing will be obtained.

Sufficient heating capacity should be provided to consistently maintain the design sludge temperature considering insulation provisions and ambient cold weather temperature conditions. Where digester gas is used for other purposes, an auxiliary fuel may be required. The design operating temperature should be 35°C (95°F) for optimum mesophilic digestion. Operating temperature may have to be elevated to 38°C to 40°C to achieve the *E. coli* standards required for agricultural land application of sewage biosolids especially for digesters that may have short-circuiting or dead volume problems.

The provision of standby heating capacity or the use of multiple units sized to provide the heating requirements should be considered unless acceptable alternative means of handling raw sludge are provided for the extended period that digestion process outage is experienced due to heat loss.

A suitable automatic mixing valve should be provided to temper the boiler water with return water so that the inlet water to the removable heat jacket or coil in the digester can be held below a temperature at which caking will be accentuated. Manual control should be provided by suitable bypass valves.

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at a minimum 82°C (180°F) to minimize corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, low gas pressure, or excessive boiler water temperature or pressure. The boiler water chemical quality should be checked for suitability for this use. Refer to *Section 8.7.2 - Water Supply* for required break tank for indirect water supply connections.

Boiler water pumps should be sealed and sized to meet the operating conditions of temperature, operating head and flow rate. Duplicate units should be provided.

Thermometers should be provided to indicate inlet and outlet temperatures of the sludge, hot water feed, hot water return and boiler water.

Controls necessary to ensure effective and safe operation are required. Provision for duplicate units in critical elements should be considered.

16.2.5.2 Gas Collection and Handling

All portions of the gas system including the space above the tank liquid level, storage facilities and piping should be so designed that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under pressure. All enclosed areas where any gas leakage might occur should be adequately ventilated.

Anaerobic digestion systems produce digester gas which has methane as its main constituent. To safeguard anaerobic digester and gas handling system design, the designer should follow the requirements of the Canadian Gas Association (CGA) *Code for Digester Gas (and Landfill Gas) System Installation*, CAN/CGA-B105-M93 (1993).

The Ontario Technical Standards and Safety Authority (TSSA), if requested, will carry out a review of anaerobic digester gas systems design on a fee-for-service basis prior to construction. Certification of these systems will only be granted following inspection of the constructed works by the TSSA.

To provide gas storage volume and to maintain uniform gas pressures, a separate gas storage sphere should be provided or at least one digester cover should be of the gas holder floating type. If only one floating cover is provided it should be on the secondary digester. Pressure and vacuum relief valves and flame arrestors, adequately protected from the elements, should be provided. Access manholes and sampling wells should be provided on the digester covers.

Steel is the most commonly used material for digester covers. However, other properly designed and constructed materials are used such as fiberglass and concrete.

All necessary safety facilities should be included where digester gas is produced. Pressure and vacuum relief valves and flame arrestors together with automatic safety shut off valves should be provided and protected from freezing. Water seal equipment should not be installed. Safety equipment and gas compressors should be housed in a separate room with at least one exterior door.

16.2.5.3 Gas Piping

Gas piping should have a minimum diameter of 100 mm (4 in). A smaller diameter pipe at the gas production meter is acceptable. Gas piping should slope to condensation traps at low points. The use of float-controlled condensate traps is not permitted. Condensation traps should be protected from freezing. The Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation*, CAN/CGA-B105-M93, 1993 or its amendments need to be reviewed and the applicable code requirements met.

16.2.5.4 Gas Appurtenances

Tightly fitted self-closing doors should be provided at connecting passageways and tunnels, which connect digestion facilities to other facilities to minimize the spread of gas. Piping galleries should be ventilated in accordance with the Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation,* CAN/CGA-B105-M93, 1993.

Gas burning boilers and engines should be located in well-ventilated rooms. Such rooms would not ordinarily be classified as a hazardous location if isolated from the digestion gallery. Gas lines to these units should be provided with suitable flame traps.

Electrical equipment, fixtures and controls in places enclosing and adjacent to anaerobic digestion appurtenances, where hazardous gases may accumulate, should comply with the *Electrical Safety Code* for Class I, Zone 1 (old Division 1), Group D locations (O. Reg. 164/99 made under the *Electricity Act, 1998*), Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation,* CAN/CGA-B105-M93, 1993 and the National Fire Protection Association's (2003) *Fire Protection in Wastewater Treatment and Collection Facilities,* US NFPA 820.

Waste gas burners should be readily accessible and should be located at least 15 m (50 ft) away from the digester perimeter and any plant structure. Waste gas burners should be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions. All waste gas burners should be equipped with automatic ignition such as a pilot light or a device using a photoelectric cell sensor. Pilot light should be either natural or propane gas type.

Gas piping should be sloped at a minimum of 2 percent up to the waste gas burner with a condensate trap provided in a location not subject to freezing.

The ventilation rate for enclosed areas without a gastight partition from the digestion tank or areas containing gas compressors, sediment traps, drip traps, gas scrubbers, or pressure regulating and control valves, if continuous, should be at least 12 complete air changes per hour.

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment should be provided with forced ventilation in accordance with the Canadian Gas Association (2005) *Code for Digester*

Gas and Landfill Gas Installation, CAN/CGA-B105-M93, 1993 and the National Fire Protection Association's (2003) *Fire Protection in Wastewater Treatment and Collection Facilities*, US NFPA 820.

16.2.5.5 Gas Metering

A gas meter with bypass should be provided to meter total gas production for each active digestion unit. Total gas production for two-stage digestion systems operated in series may be measured by a single gas meter with proper interconnected gas piping.

Where multiple primary digestion units are used with a single secondary digestion unit, a gas meter should be provided for each primary digestion unit. The secondary digestion unit may be interconnected with the gas measurement unit of one of the primary units. Interconnected gas piping should be properly valved with gastight valves to allow measurement of gas production from either digestion unit or maintenance of either digestion unit.

Gas meters are generally of the orifice plate, turbine, thermal dispersion, or vortex type. Positive displacement meters should not be used. The meter should be designed specifically for contact with corrosive, moist and dirty gases.

16.2.5.6 Supernatant Withdrawal

Where supernatant separation is to be used to concentrate sludge in the digester units and increase digester solids retention time, the design should provide for ease of operation and positive control of supernatant quality.

Supernatant piping should not be less than 150 mm (NPS-6) in diameter.

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. An unvalved vented overflow should be provided. The emergency overflow should be piped to an appropriate point and at an appropriate rate to the STP liquid train or sidestream treatment units to minimize the impact on process units. The design of the overflow should prevent digester gas migration to other process areas. Supernatant flow measurement should be provided.

On fixed-cover tanks the supernatant withdrawal level should preferably be selected by means of interchangeable extensions at the discharge end of the piping.

A fixed-screen supernatant selector or similar type device should be limited for use in an unmixed secondary digester unit. If such supernatant selector is provided, provision should be made for at least one other draw-off level located in the supernatant zone of the tank, in addition to the unvalved emergency supernatant draw-off pipe. High-pressure backwash facilities should be provided.

Provision should be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 40 mm (1½ in) in diameter and should terminate at a suitably sized sampling sink or basin meeting the Code requirements. Special designed sampling valves that ensure a representative sample, can be used in lieu of a sampling sink.

Supernatant return facilities should be designed to alleviate adverse hydraulic and organic loadings effects on sewage treatment plant liquid train operations. If nutrient removal (e.g. phosphorus) is to be accomplished at an STP, then a separate supernatant sidestream treatment system may be provided.

16.2.5.7 Sludge Production

For calculating design sludge handling and disposal needs, sludge production values from a two-stage anaerobic digestion process should be based on a maximum solids concentration of 4 percent without additional thickening. Facility sizing using population equivalent (P.E.) should not be used as the primary basis for design. Process calculation based on raw sewage treated and process type should form the basis for designing of needed facilities where P.E. is used for rough estimation of needed capacities. Estimates should also consider other solids accepted by the STP including (but not limited to) septage and water treatment plant residuals. Where P.E. is used for rough estimation, the approximate sludge production values on a dry weight basis should be at least 0.07 kg/(P.E.·d) (0.15 lb/P.E./day) for a conventional activated sludge plant with phosphorus removal.

16.2.6 Thermophilic Anaerobic Digestion

Thermophilic anaerobic digestion is in principle similar to anaerobic mesophilic digestion, except that thermophilic digestion occurs between 50 to 60ºC (122 to 140ºF). Because biochemical reaction rates increase with increased temperatures, thermophilic digestion is faster than mesophilic digestion for the same volatile solids reduction. The higher temperature at which thermophilic digestion takes place allows for increased pathogen destruction.

16.2.6.1 Process Variables

Similar to mesophilic digestion, the following are the four main process variables to be considered in the design and operation of thermophilic anaerobic digestion:

- Solids loading rate;
- Solids retention time (SRT);
- Hydraulic retention time (HRT);
- Temperature; and
- pH.

16.2.6.2 Design Considerations

Thermophilic anaerobic digestion generally has the same design considerations as anaerobic mesophilic digestion, specifically:

- Digester shape;
- Digester cover and bottom;
- Digester volume;
- Mixing system;
- Heating system; and
- Gas collection, storage and use.

16.2.6.3 Operational Considerations

The same operational considerations for mesophilic digestion generally apply to thermophilic digestion, such as:

- Feeding and withdrawal;
- Temperature control;
- pH and alkalinity monitoring; and
- Odour control.

Because of the higher sensitivity of thermophilic bacteria to temperature changes and potential process upsets, temperature control, pH and alkalinity monitoring are important. Thermophilic anaerobic digestion produces a higher concentration of volatile fatty acids (1,000 to 2,000 mg/L) than mesophilic digestion. The higher operating temperature in thermophilic digesters tends to suppress scum and foam formation, so that scum and foam control is often less problematic than in mesophilic digesters.

16.3 AEROBIC SLUDGE DIGESTION

16.3.1 General

The aerobic sludge digestion system should include provisions for digestion, supernatant separation, sludge concentration and sludge storage. These provisions may be accomplished by separate tanks or processes, or within the digestion tanks.

Multiple digestion units capable of independent operation are desirable and should be provided in STPs where the design average daily flow exceeds 380 m^3/d (0.1 mUSgd). Two stages with a minimum of one digester in each stage should be provided. If economics permit or plant size dictates, more than one digester can be used. Plants not having multiple units should provide alternate sludge handling and disposal methods. A loading rate of 1.6 $kg/(m^3 \cdot d)$ [100 lb/(1000 ft^3·d)] volatile solids based upon first stage volume only should be provided.

Tank design is generally open and may be common wall or earthen bermed to minimize heat loss. Aerobic digesters may be covered to minimize heat loss for colder temperature applications. Tank depths of 3.6 to 4.6 m (11.8 to 15.1 ft) are suggested. The tanks and piping should be designed to permit sludge addition, withdrawal and supernatant decanting from various depths to and from both the primary and secondary digesters.

16.3.2 Tank Design Capacity

Digestion tank capacities are based on a solids concentration of 2 percent with supernatant separation performed in a separate tank and providing sufficient tank volume based on process engineering calculations using sewage and treatment characteristics. Volumes are based on digester temperatures of 10°C (50°F).

Sizing should be designed to achieve a minimum SRT of 45 days, including both digester stages and the SRT of the activated sludge treatment process. It is recommended that 2/3 of the total digester volume be in the first stage and 1/3 be in the second stage.

If supernatant separation is performed in the digestion tank, a minimum of 25 percent additional tank volume is required. These capacities should be provided unless sludge thickening facilities are used to thicken the feed solids concentration to greater than 2 percent. If such thickening is provided, the digestion volumes may be decreased proportionally; however, excess thickening may result in the occurrence of unintended autothermal digestion resulting in foam, scum and odour that requires control.

If primary sludge is to be included for digestion, minimum SRT and air requirements may have to be increased based on specific process calculations.

Actual storage requirements will depend upon the ultimate disposal operation. Minor additional storage requirements may be made up in the second stage digester, but if major additional storage volumes are required, separate on-site or off-site storage facilities should be considered to avoid the power requirements associated with operating greatly oversized aerobic digesters. The *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management Act* should be reviewed for specific quality and seasonal storage requirements for a land application program.

Designers should be cautioned that aerobically digested sludges have a greater odour producing potential then anaerobically digested sludge, if septicity occurs.

16.3.3 Mixing

Aerobic digesters should be provided with mixing equipment that can maintain solids in suspension and ensure complete mixing of the digester contents.

16.3.4 Air Requirements

Sufficient air should be provided to keep the solids in suspension and maintain dissolved oxygen between 1 mg/L and 2 mg/L. For minimum mixing and oxygen requirements, an air supply of 0.5 L/(m^3·s) (30 cfm/1000 ft^3) should be provided with the largest blower out of service. If diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service. Air supply to each tank should be separately valved to allow aeration shut-down in either tank. The diffuser type should not be susceptible to plugging during frequent shutdown periods. All diffuser drop pipes should be able to withstand impact of ice masses that may form in the tankage in winter and should allow for easy removal for diffuser maintenance.

If mechanical turbine aerators are used, at least two turbine aerators per tank should be provided to permit continuity of service. A minimum bottom velocity of 0.25 m/s (0.82 ft/s) should be maintained while aerating. Mechanical aerators are not recommended for use in aerobic digesters where freezing conditions will cause ice build-up on the aerator and support structures.

16.3.5 Supernatant Separation and Scum and Grease Removal

Facilities should be provided for effective separation or decanting of supernatant. Separate facilities are recommended; however, supernatant separation may be accomplished in the digestion tank provided additional volume is provided (*Section 16.3.2 - Tank Design Capacity*).

The supernatant draw off unit should be designed to prevent recycle of scum and grease back to the STP liquid train process units. Facilities should be provided for the effective collection of scum and grease from the aerobic digester for final disposal to prevent long-term accumulation and potential discharge in the effluent. Provision should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

16.3.6 High Level Emergency Overflow

An unvalved high level overflow and necessary piping should be provided to return digester overflow to the head of the sewage treatment plant or to the secondary treatment process in case of accidental overfilling. Design considerations related to the digester overflow should include waste sludge rate and duration during the period that the sewage treatment plant is unattended, potential effect on plant process units, discharge location of the emergency overflow and potential discharge of suspended solids in the STP effluent.

16.3.7 Sludge Production

For calculating design sludge handling and disposal needs, sludge production values from aerobic digesters should be based on a maximum solids concentration of 2 percent without additional thickening. Facility sizing using population equivalent should not be used as the primary basis for design. Process calculations based on raw sewage treated, process type, chemical addition, septage solids and water treatment plant waste solids impacts, where applicable, should form the basis for design of needed facilities where P.E. is used for rough estimation of needed capacities.

Where P.E. is used for rough estimation, the approximate sludge production values on a dry weight basis should be at least 0.05 kg/(P.E.·d) (0.11 lb/ P.E./d) for an extended aeration STP with phosphorus removal.

16.3.8 Digested Sludge Storage Volume

Sludge storage should be provided in accordance with the *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management Act* to accommodate sludge production volumes and as an operational buffer for unit outage and adverse weather conditions for biosolids destined for land application. Designs utilizing increased SRT in the activated sludge treatment process as a means of storage should be avoided.

The designer should refer to *Chapter 18 - Sludge Storage and Disposal* for more detailed information.

16.4 AUTOTHERMAL THERMOPHILIC AEROBIC DIGESTION

16.4.1 General

Thermophilic aerobic digestion is in principle similar to mesophilic aerobic digestion, except that thermophilic digestion occurs between 50 to 70°C (122 to 158°F). Because biochemical reaction rates increase with increased temperatures, thermophilic digestion is faster than mesophilic digestion for the same, or greater volatile solids reduction. The higher temperature at which thermophilic digestion takes place allows for increased pathogen destruction. Higher solids loading and lower air flows allow for the excess energy from the exothermic reaction to raise the temperature of the solids to required levels without an external heat source. Mechanical energy is supplied for mixing in lieu of utilizing air mixing.

Retaining the heat from the biological process is vital to this process. The tank design should consider insulation, below grade construction or other means to control heat loss from the system. The effects of heat and corrosion on the tank (and cover) structures should be considered in the design.

The total solids concentration to the digesters should be 4 to 6 percent, and the volatile solids should be at least 2.5 percent. Volatile solids (VS) could range from 2.5 to 5 percent. Other loading rates are possible and should be established based on detailed calculations and on a site-specific basis, provided those rates can maintain adequate process temperatures.

Tanks should provide a freeboard of 1.0 to 2.0 m (3 to 6 ft). Provisions for draining and cleaning the tanks should be considered.

Single-Stage Systems

Single-stage Autothermal thermophilic aerobic digestion (ATAD) units operate with the entire process being completely contained within one tank. Multiple tanks may be provided but are often operated as parallel units and may be loaded on alternate days to allow for the appropriate isolation period. Continuous or semi-batch feeding is more common in these systems and allows for the distribution of the aeration demands throughout the entire process cycle. Process temperatures for single-stage units should be maintained at 50 to 70^{o}C (122 to 158^{o}F). Detention time should be a minimum of 10 days. The foam blanket should be controlled and optimized but not eliminated. Energy requirements for mechanical mixing may be in the range of 50 to 150 kW/1000 m^3 (2 to 6 hp/1000 ft^3) of active digester volume. Air requirements may range from 10 to 50 m^3/min/1000 m^3 (10 to 50 cfm/1000 ft^3) of active digester volume to achieve 30 to 45 percent VS destruction and are often operated on variable frequency drives. Air requirements should be based on loading rate and degree of VS destruction. Adjustments to the air requirements for altitude and loading rates should be considered.

Multiple-Stage Systems

The multiple-stage ATAD process consists of two or more stages. It is good practice to provide tankage, piping and pumping facilities so that at least two tanks are available. The piping should allow transfer of biosolids from one tank to another to allow for maintenance and continuous digester operation. Process temperatures for the first stage should be maintained at 35 to 50°C (95 to 122°F). Second stage temperatures should be maintained at 50 to 65°C (122 to 149°F). Detention time should be a minimum of 10 days. The foam blanket should be controlled and optimized but not eliminated. The dissolved oxygen (DO) level in the liquid should be maintained at 2 mg/L and should not be less than 1 mg/L.

Energy requirements for mechanical mixing should be in the range of 79 to 105 kW/1000 m^3 (3 to 4 hp/1000 ft^3) of active digester volume. Air requirements should be at least 70 m^3/min/1000 m^3 (70 cfm/1000 ft^3) of active digester volume to achieve 30 to 45 percent VS destruction. Adjustments to the air requirements for altitude and loading rates should be considered.

16.4.1.1 Operational Considerations

The same operational considerations for mesophilic aerobic digestion generally apply to thermophilic digestion, including:

- Feeding and withdrawal;

- Temperature control;
- pH;
- DO;
- Oxidation-reduction potential;
- Foam control; and
- Odour control.

The thermophilic bacteria are less sensitive to process changes and are not prone to process upsets. Temperature control and solids monitoring are important. The higher operating temperature in the thermophilic aerobic digester in combination with aeration tends to produce more foam, so foam control is an important design consideration. Additionally, given the high temperature and the increased ammonia from the volatile solids destruction, ammonia and other compounds may be given off in the off gas. Odour collection and control is therefore an important consideration in the design.

16.5 OTHER SLUDGE TREATMENT METHODS

The following sludge treatment methods are generally employed to further stabilize biosolids and/or reduce volume.

16.5.1 Alkaline Stabilization

Alkaline material may be added to liquid primary and/or secondary sludges for sludge stabilization in lieu of digestion facilities, to supplement existing digestion facilities, or for interim sludge handling. There is no direct reduction of organic matter or sludge solids with the high pH alkaline stabilization process. There is actually an increase in the mass of dry sludge solids. Without supplemental dewatering, additional volumes of sludge will be generated. The design should account for the increased sludge quantities for storage, handling, transportation, disposal methods and associated costs.

Alkaline material should be added to liquid sludge to produce a homogeneous mixture with a minimum pH of 12 after 2 hours of vigorous mixing. To achieve stabilization sufficient alkaline material should be added to the liquid sludge to retain a homogeneous mixture with a minimum pH of 12 after 72 hours. Material should be provided to maintain the pH of the sludge during interim sludge storage periods.

Other proprietary alkaline stabilization systems using various alkaline reagents which include a pasteurization process are available to produce biosolids from sludge.

16.5.1.1 Odour Control and Ventilation

Odour control facilities should be provided for sludge mixing and treated sludge storage tanks when located within 0.8 km (0.5 mile) of residential or commercial areas. Air pollution control design objectives should be met for various types of air scrubber units. Ventilation is required for indoor sludge mixing, storage or processing facilities. (*Section 7.2.10 - Safety Ventilation)*.

16.5.1.2 Mixing Tanks and Equipment

Mixing tanks may be designed to operate as either a batch or continuous flow process. A minimum of two tanks should be provided, of adequate size to provide a minimum of 2 hours contact time in each tank.

The following items should be considered in determining the number and size of tanks:

- Peak sludge flow rates;
- Storage between batches;
- Dewatering or thickening performed in tanks;
- Repeating sludge treatment due to pH decay of stored sludge;
- Sludge thickening prior to sludge treatment; and
- Type of mixing device used and associated maintenance or repair requirements.

Mixing equipment should be designed to provide vigorous agitation within the mixing tank, maintain solids in suspension and provide for a homogeneous mixture of the sludge solids and alkaline material. Mixing may be accomplished either by diffused air or mechanical mixers. If diffused aeration is used, an air supply of 0.5 L/(m^3·s) (30 cfm/1000 ft^3) of mixing tank volume should be provided with the largest blower out of service. When diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service. If mechanical mixers are used, the impellers should be designed to minimize fouling with debris in the sludge. Manufacturers' specifications should be reviewed for mixing power required for mechanical mixers. An approximate minimum requirement for liquid sludge and lime slurries is to provide a bulk fluid velocity of 0.13 m/s (0.42 ft/s) and an impeller Reynolds number greater than 1,000.

Consideration should be made to provide continuity of service during freezing weather conditions.

16.5.1.3 Chemical Feed and Storage Equipment

Alkaline material is caustic in nature and can cause eye and tissue injury. Equipment for handling or storing alkaline material should be designed for adequate operator safety. Storage and feed equipment should be sealed as airtight as practical to prevent contact of alkaline material with atmospheric carbon dioxide and water vapor and to prevent the escape of dust material. All equipment and associated transfer lines or piping should be accessible for cleaning.

16.5.1.4 Feed and Slaking Equipment

The design of the feeding equipment should be determined by the STP size, type of alkaline material used, slaking required and operator requirements. Equipment may be either of manual batch or automated type. Automated feeders may be of the volumetric or gravimetric type depending on accuracy, reliability and maintenance requirements. Manually operated batch slaking of quicklime (CaO) should be avoided unless adequate protective clothing and equipment are provided. At small plants, use of hydrated lime [$Ca(OH)_2$] is recommended over quicklime due to safety and labour-saving reasons. Feed and slaking equipment should be sized to handle a minimum of 150 percent of the peak sludge flow rate including sludge that may need to be retreated due to pH decay. Duplicate units should be provided.

16.5.1.5 Chemical Storage Facilities

Alkaline materials may be delivered either in bag or bulk form depending upon the amount of material used. Material delivered in bags should be stored indoors and elevated above floor level. Bags should be of the multi-wall moisture-proof type. Dry bulk storage containers should be as airtight as practical and should contain a mechanical agitation mechanism. Storage facilities should be sized to provide a minimum 30-day supply.

16.5.1.6 Sludge Storage and Disposal

Refer to *Chapter 18 - Sludge Storage and Disposal* for general design considerations for sludge storage facilities. In addition, the design should incorporate the following considerations for the storage of high pH stabilized sludge:

Liquid Sludge

High pH stabilized liquid sludge should not be stored in a lagoon. The sludge should be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or treatment. Provisions should be made for adding alkaline material in the storage tank. Mixing equipment should be provided in all storage tanks.

Dewatered Sludge

On-site storage of dewatered high pH stabilized sludge should be limited to 30 days. Provisions for rapid treatment or disposal of dewatered sludge cake stored on-site should be made in case of sludge pH decay. Weather protection for dewatered sludge cake should be provided for long-term storage.

Off-Site Storage

There should be no off-site storage of high pH stabilized sludge unless specifically designed for such storage.

Disposal

Immediate sludge disposal methods and options are recommended to reduce the sludge inventory on the STP site and the amount of sludge that may need to be retreated to prevent odours if sludge pH reduction occurs. If the land application disposal option is utilized for high pH stabilized sludge, the sludge should be incorporated into the soil during the same day of delivery to the site.

16.5.2 Thermal Drying

Thermal drying is the process of evaporating water from sludge or biosolids by the addition of heat. Complete drying typically results in a product with 5 to 10 percent moisture content, corresponding, from a typical liquid sludge with 95 percent moisture content, to an approximate 20-fold volume reduction. Dewatering is usually a prerequisite intermediate step in the drying process to reduce energy costs for evaporation of moisture (refer to Chapter 17).

During drying, sludge or biosolids undergo several structural changes as the moisture content decreases. The most critical stage is called the plastic stage when the moisture content is between 40 to 60 percent dry solids (DS). In this stage, the dried product becomes sticky and difficult to manipulate. The power input required to move the product through this phase to higher concentrations is high.

Dryers are classified on the basis of:

- The predominant method of transferring heat to the solids (convection, conduction, radiation or a combination of these);
- The method of transition through the plastic phase;
- Whether drying and pelletization occurs in one or two steps; and
- Whether the biosolids are partially (i.e., <90 percent DS) or completely (i.e., >90 percent DS) dried.

The main benefits of drying sludge thermally can be summarized as follows:

- Increased pathogen destruction is achieved;
- Storage of dried sludge requires less volume and is easier to handle;
- Transportation costs are reduced;
- The final product can be marketed more easily as a fertilizer or soil conditioner;
- Dried sludge has a higher fuel value and can be incinerated or thermally converted; and
- Sludge drying increases the number of final disposal or utilization options.

Two general alternatives for thermal drying are direct and indirect systems. Thermal drying equipment is proprietary; the designer should consult the manufacturer to obtain specific design parameters.

16.5.2.1 Direct (Convection) Dryer Process Alternatives

In convection dryers the wet sludge is in direct contact with the heat transfer medium, which is usually a hot gas. Direct dryer designs typically include:

- Rotary drum dryers; and
- Fluidized bed dryers.

Rotary Dryers

Rotary dryers have found successful application in municipal biosolids facilities. Rotary dryers consist of a horizontal cylindrical steel drum, rotating at 5 to 25 rpm. The wet sludge/biosolids are mixed with an amount of dried product at the feed point. Flue gases from a burner flow co-currently and in direct contact with the biosolids. The mixture of biosolids and hot gases is conveyed to the discharge end of the drier where the dry product is separated from the gas and vapour mixture.

The temperature of the hot gas at the inlet of the drum is typically between 450 to 500°C (842 to 932°F) and the temperature of the product is approximately 100 to 140°C (212 to 284°F). The oxygen content at the dryer outlet is between 15 and 17 percent. The flue gas and vapour mixture is sent to a condenser and the flue gases and non-condensables should be treated in an odour control unit.

There are three main disadvantages with these types of dryers: the high oxygen content in the drum which presents fire and explosion risks, the large volume of gas that needs to be treated in an odour control unit and the energy losses from the large stack required. To address these disadvantages, some manufacturers implement air/vapour recirculation systems with heat exchangers.

Fluidized Bed Dryers

In fluidized bed systems, the biosolids are fluidized when brought into contact with hot gases moving upward. These are vertically mounted systems and in recent designs the hot gases are recirculated in a closed loop. Wet biosolids are mixed with dried product, enter at the top of the chamber and sink to the bottom. As the product dries its density decreases and as a result it occupies the highest part of the dryer. The dried product is discharged through an overflow and the gases are directed to a cyclone separator and an odour control unit. The cyclone captures the dust created by the attrition of the particles caused by fluidization. Fluidized bed dryers tend to be sensitive to variations in sludge composition because of its effect on the fluidization process. The heat exchangers incorporated in the chamber suffer from abrasion. The system is considered to have high power requirements.

16.5.2.2 Indirect (Conduction) Dryer Process Alternatives

In conduction dryers a solid retaining wall separates the wet sludge/biosolids from the heat transfer medium, which is usually steam or another hot fluid. Indirect (conduction) dryers include:

- Thin film dryers;
- Disc dryer;
- Paddle dryers;
- Vertical dryers;
- Pelletizers; and
- Multiple-effect evaporation dryers.

A brief description of the most common indirect dryer systems employed for municipal sludge/biosolids follows.

Thin Film Dryers

This is an example of a drying system that dries biosolids through the plastic phase without dry product recirculation. It is a horizontal system where biosolids are introduced in a fixed shell containing a rotating shaft. The material is spread onto the wall where it forms a thin film on a steam or thermal oil heated jacket. Blades mounted on the shaft scrape the product and force it across the dryer to the discharge end.

The main disadvantage of this type of drying system is the large amount of mechanical wear exerted by the dried product when it is above 80 percent DS.

Disc Dryers

Disc dryers are composed of heated hollow discs set one after the other in parallel along a rotor. The discs and rotor are enclosed in a fixed shell. Biosolids fill the shell and submerge the discs and rotor. There are scrapers attached to the encasing shell extending inward until just above the rotor shaft and the discs are equipped with large paddles, which control the residence time of the product. Disc dryers can be used for partial or complete drying. If used for complete drying, dried product is mixed with wet feed before entering the dryer. This configuration is subject to heavy mechanical wear.

Paddle Dryers

Paddle dryers have a similar configuration to disc dryers. Hollow wedge-shaped, self-cleaning blades take the place of the discs and casing. The rotor speed is low and the residence time is high. Paddle dryers are subject to similar wear problems as disc dryers when used for complete drying.

16.5.2.3 Radiation Dryer Process Alternatives

In radiation dryers, infrared lamps, electric resistance elements or gas-heated incandescent refractories supply the energy required to heat the wet sludge/biosolids and evaporate moisture.

16.5.2.4 Solar Thermal Drying

Solar thermal drying within a greenhouse is a thermal dewatering and drying process that uses solar energy as the primary energy source. This process is applicable to municipal sludge/biosolids. Proprietary systems are available that utilize this technology.

In solar drying, vapour pressure differences result in evaporation of moisture to the atmosphere. The transparent cover provides insulation and transmission of solar radiation and is constructed to withstand wind and snow loads. Liquid or dewatered sludges are spread out on the floor of the drying pens and are mixed and passively aerated through the mixing action. Automated ventilation and optional heating system should be provided.

Solar drying can provide a more controlled environment, however the technology requires a significant amount of land. It is modular and can be decentralized. Solar drying facilities can handle liquid sludge from 3 to 10 percent dry solids (DS) and dewatered sludges from 10 to 40 percent DS. The treatment process provides dewatering and drying. The process can achieve a substantial volatile solids and pathogen reduction depending on the drying time. Drying times of three months during winter operation without external energy, and shorter periods during summer conditions, are typical.

16.5.3 Pelletization of Dried Sludge

Pelletization of dried sludge can be used, especially if the dry product is to be used as a fertilizer or soil supplement. Pelletization may be accomplished in one step with drying or it may follow as a separate step. In the former case, dewatered sludge is first sent through a pelletizer where it is transformed into a pellet. This procedure gives cohesion to the sludge and creates a large external surface area that accelerates the drying process.

16.5.4 Indirect Vertical Dryer

Another process is an indirect vertical dryer with a number of heated trays constructed one above the other inside a cylindrical shell. The sludge/biosolids dry as they contact the heated trays. Recycled pellets are coated with a thin layer of incoming wet material and introduced to the dryer at the top. As they move from the top to the bottom trays they dry out and are finally transported to a separation hopper where they are sorted for size. Pellets are recycled 5 to 7 times, growing in size with each pass through the dryer, until they reach the desired diameter at which stage they are separated from the recycling stream and sent to the storage facility.

Pelletization is provided for the following reasons:

- Storage of dried sludge/biosolids pellets reduces the risk of fire and glowing, which is higher for dried sludge dust; and
- Handling of dried sludge pellets is easier and poses less danger to the environment and the personnel in contact with it.

16.5.4.1 Technical Considerations

For each of the dryer categories described above, there are specific manufacturer technical and design considerations. However, the following factors and considerations apply to all types of dryers and play the most important role in the dryer selection and sizing.

- The desired moisture content of the wet and dried sludge/biosolids will affect dryer selection;
- The amount of flexibility required in the design to accommodate varying sludge/biosolids characteristics;
- Mechanical dewatering is a requirement prior to drying;
- Continuous or batch drying operations affect dryer size;
- Storage requirements for wet and dried sludge/biosolids are an important consideration;
- Condensate from air recycle streams should be considered;
- Dust may be a hazard during processing or if the dried biosolids are stored in large volumes; dust creation should be prevented to avoid ignition and explosive conditions;
- Energy sources for the dryer may be natural gas or fuel oil; because of the large amounts of energy required, recovery of heat from the exhaust gases should be considered; in addition, future energy costs should be considered;
- Safety requirements, especially prevention of risk of fire or explosion; sludge dried and stored in reducing conditions may dry exothermically and potentially lead to autogenous combustion (the National Fire Protection Association's (2003) *Fire Protection in Wastewater Treatment and Collection Facilities,* US NFPA 820 requirements should be reviewed); and
- Consideration should be given to odour control especially if unstabilized sludge/biosolids are dried (if rewetted, odours will be emitted). (*Section 4.4 - Odour Control and Abatement Measures*).

16.5.5 Composting

Composting is a biological process in which organic material undergoes biological degradation to a stable end product called humus. Composting has received attention as an alternative for enhanced stabilization and utilization of biosolids for a number of potential beneficial uses.

The Canadian Food Inspection Agency (CFIA) regulates the use of compost in accordance with the Fertilizers Regulations (C.R.C., c. 666) made under the *Fertilizers Act* (R,S.C. 1985, c. F-10). Canadian Council of Ministers of the Environment (CCME) *Guidelines for Compost Quality* (2005) specifies criteria for product safety and quality: foreign matter, maturity, pathogens and trace elements. The Standards Council of Canada (SCC) provides voluntary National Standard of Canada - *Organic Soil Conditioners - Composts.*

The ministry guidelines *Interim Guidelines for the Production and Use of Aerobic Compost in Ontario* provides guidance applicable to the establishment and operation of composting facilities in Ontario.

The designer should be familiar with the current requirements of these regulations and guidelines and consult with the ministry regarding site-specific design and operating criteria related to buffer zones, storage of composting material, runoff or leachate control, odour control and other process issues.

Composting is accomplished under aerobic conditions. The self-heating aerobic process attains temperatures in the pasteurization range of 50 to 70°C (122 to 158°F). This results in the inactivation of pathogens and the production of well-stabilized compost that can be stored and has minimal odour. The high quality biosolids product can be used beneficially as a soil conditioner or organic fertilizer supplement.

Maintenance of a minimum temperature of 55°C (131°F) for at least three days can achieve virtually complete inactivation of pathogens within in-vessel and aerated static pile systems. Composting is a sludge processing technology that, depending on process design, can treat dewatered undigested and/or digested sludge and potentially produce a “pathogen-free” biosolids product. Dewatering is to provide at least 18 percent solids concentration prior to addition of bulking agents. This could defer or eliminate the need for future digester upgrades and expansions and can represent a flexible option as part of a diversified biosolids management program. Additional volatile solids destruction and degradation of persistent organic substances in digested biosolids may be possible.

The mass of compost product is typically about one-half of the mass of wet dewatered sludge that is added to the process. However, there is little change in the volume, as the product is less dense than the wet sludge.

Methods for composting include in-vessel, windrow and static aerated pile composting. Table 16- 2 summarizes key requirements.

Table 16- 2 - Summary of Composting Requirements

Parameter	Aerated Static Pile	In-Vessel	Windrow
Overall Active Composting Time (days)	21-28	10-21	21-28
Time at ≥ 55ºC (days)	3	3	15
Overall Time Including Curing (days)	50-80	50-80	50-80
Turning Times (#)	-	-	5
Solids Content After Dewatering (%)	≥18	≥18	≥18
Minimum Supply Mixture Solids, Including Bulking Agent (%)	40	35	40

Three separate stages of microbial activity occur during the composting process:

- Initial mesophilic stage, during which temperatures within the pile increase from ambient to about 40ºC (104°F);
- Thermophilic stage, caused by the heat generated through conversion of organic matter to carbon dioxide and water vapour, where temperatures can range from 40 to 70ºC (104 to 158°F); and
- Cooling stage associated with reduced microbial activity as composting approaches completion (i.e., curing).

Composting under aerobic conditions, depending on the system design, involves the following steps:

- Mixing of dewatered sludge with a bulking agent or amendment to ensure an adequate mixture porosity for proper aeration, structural integrity, acceptable mixture density, reduced bulk moisture content and to provide supplemental carbon to adjust the energy balance and carbon-to-nitrogen ratio;
- Aeration and/or agitation of the mixture to promote the aerobic microbiological decomposition reactions (i.e., active composting); and
- Curing of the compost to complete the stabilization process.

In addition to providing the required oxygen for organics degradation, aeration and agitation facilitate the removal of exhaust gases, water vapour and heat. The rate of aeration may be used to control process temperature and the rate of drying. Drying during the composting process can produce solids concentrations of 50 to 55 percent.

Product curing, which follows active composting, may be preceded or followed by screening. The overall detention time for composting and curing is typically between 50 to 80 days. If feasible, the bulking agent is recovered by screening for reuse. An area for temporary storage of the final stabilized product is usually provided at the composting site.

16.5.5.1 Process Alternatives

The following are the major available process options for sludge composting:

- Aerated static pile;
- Windrow; and
- In-vessel systems.

Aerated Static Pile

In the aerated static pile process, the mixture of dewatered cake and coarse bulking agent is placed over a porous bed (i.e., a grid of closed and perforated piping). Air is supplied to each pile by a dedicated blower and piping and is drawn downward or forced upward through the mixture. The pile is covered with an insulating blanket of wood chips or screened compost. The active composting period is 21 to 28 days.

Small applications can consist of a number of individual piles whereas larger applications can involve a continuous pile that is divided into sections representing the contribution of each day. New facilities are typically covered and some are fully enclosed for reduced odour and for improved process control.

Windrow

Windrows consist of long narrow parallel piles of the mixture through which aeration is achieved by natural convection and diffusion. In the aerated windrow process, supplemental forced aeration through underlying air channels is used. The windrow is remixed periodically by a turning mechanism to facilitate air movement and moisture release. Windrow operations are covered or enclosed systems. The active composting period is 21 to 28 days.

In-Vessel Systems

In-vessel systems for active composting are enclosed and mechanized processes, comprising a reactor(s) and conveyors that offer an increased degree of process and odour control. The systems are compact and can be highly automated, including programmable logic controller (PLC)-based automatic control systems. The control of environmental conditions such as air flow, temperature, moisture and oxygen concentration permits shorter composting times.

The mixture of dewatered sludge, amendment and recycled compost is fed into one end of a tunnel, silo, or channel of the in-vessel process and moves continuously towards the discharge end. Air supplied by blowers is forced through this mixture which may be periodically agitated.

The three typical in-vessel reactor designs include:

- Vertical plug-flow;
- Horizontal plug-flow (i.e., tunnel reactor); and
- Agitated bin reactors.

The plug-flow systems involve periodic feeding (e.g. daily) and discharge of “finished” compost from the opposite end. Unlike the plug-flow designs, the dynamic agitated bed process uses mechanical mixing during composting. Depending on the particular process or system supplier, the detention time in the reactor can vary between 10 to 21 days for active composting. Compared with static pile and windrow composting, in-vessel processes can produce a more consistent product, require less space and provide an enhanced degree of odour containment and control. Modular system designs can facilitate future expansion.

Bulking agent may be removed after completion of composting by screening to reduce mass and to recycle the bulking agent.

16.5.5.2 Technical Considerations

Process variables that can affect composting operations and performance include temperature, bed porosity, moisture content, ratio of organics to nutrients, pH, aeration levels and detention time. Parameters that can be monitored and used to control in-vessel composting processes include:

- Mixture temperature;
- Blower static pressure;
- Relative humidity of the fresh air supply;
- Relative humidity of process headspace;
- Volume of fresh air;
- Blower speed; and
- Oxygen concentration in process headspace.

A number of factors can influence selection of the most appropriate composting process for a given application. These can include:

- Characteristics of the sludge supply (e.g. solids content, degree of stabilization, if any, and loading rates);
- Type of equipment and chemicals used in upstream sludge dewatering and the consistency of the resultant cake; and
- Land availability.

Dewatered sludge cake of 18 to 25 percent solids can be mixed with bulking agent or amendment to produce the desired solids content of the feed supply.

The uniformity of the mixture with respect to porosity is critical in static pile systems and less so in windrow and in-vessel systems.

Sludge supply that is stabilized by aerobic or anaerobic digestion prior to composting can reduce the size of the composting facility due to the reduced organic solids content. In the event that the composting system is overloaded or out-of-service (e.g. for maintenance), the stabilization of sludge, prior to composting, offers the ability to directly apply biosolids to land on an interim basis. The application of biosolids to agricultural land should be designed into the program, including relevant permitting requirements (e.g. Organic Conditioning Site *Certificate of Approval*). Other considerations include:

- Composting of undigested sludge results in higher reaction rates, oxygen demand, heat generation and odour potential;
- Material being composted should be regularly mixed or turned, depending on the compost process, to prevent drying, caking and air channeling;
- Process temperature should be kept at between 55 to 65ºC (131 to 149°F) for a defined period of time until pathogen control requirements are met. For the first few days, temperature should be maintained at optimum levels of between 50 and 55ºC (122 and 131°F) to promote maximum rates of organics degradation and stabilization;
- New composting facilities typically include odour control systems for the containment and treatment of exhausts. Odour control systems can include biofilters, wet scrubbers and/or thermal oxidation for the removal of ammonia and other odour compounds; measures for odour control after winter freezing should be provided;
- Depending on process design, it may be possible to co-compost municipal STP sludge with other organic solid wastes. The latter solid wastes require pre-sorting and pulverizing prior to mixing with sludge;
- Site considerations include land availability, access, proximity to the STP, site drainage and runoff control, proximity to end users of the finished product, climatic conditions and availability of buffer zone; and
- The market for compost varies regionally based on local conditions such as land use, availability of competing soil amendment and fertilizer products, guidelines for biosolids compost and public acceptance of biosolids products.

16.5.5.3 Metals Content in Finished Compost

Metals content of finished compost affects the usability of the product and should be carefully considered during the process design to ensure a market for the final product. The ministry guidelines *Interim Guidelines for the*

Production and Use of Aerobic Compost in Ontario provides the metals concentrations criteria which finished compost needs to comply with, and should be referenced.

16.5.6 Incineration

Incineration is a unit process for sludge management that "burns" the organic matter present in the sludge. Combustion releases the heat value of the organic matter in the sludge through rapid high temperature chemical oxidation reactions and reduces the volume and weight of solid residuals (ash) for ultimate disposal. Depending upon temperature, this process can destroy, transform or reduce trace organic materials.

The incineration of wastewater sludges employs high temperature chemical reactions, typically at 700 to 870°C (1,300 to 1,600°F), which convert the organic carbon and hydrogen in the sludge and the oxygen in the combustion air into carbon dioxide and water vapour. Supplemental fuel, usually natural gas or fuel oil, is burned if required, which would use additional combustion air and produce additional carbon dioxide and water vapour. Inorganic matter in the sludge remains as a solid "ash" residue. Metals (except for relatively volatile mercury and, to a lesser extent, cadmium) are converted to stable and generally insoluble oxides within the ash.

The exhaust gases contain, in addition to carbon dioxide and water vapour, nitrogen and unconsumed oxygen from the combustion air plus a smaller amount of air pollutant emission of suspended particulates, carbon monoxide, oxides of sulphur (SO_X) and oxides of nitrogen (NO_X). These usually result when the compounds are volatilized before they are exposed to high enough temperatures to be properly oxidized. Therefore, afterburning often has to be used to destroy these volatilized compounds, especially in multiple hearth incinerators.

The heating value of the wet sludge cake is a major variable governing the design and operation of an incinerator. Sludge heating values vary depending on the types of sludges and on the form and performance of the conditioning and dewatering processes used prior to incineration. The heating values of primary and secondary sludges, both raw and digested, average about 23,300 to 25,600 kJ/kg (10,000 to 11,000 Btu/lb) of volatile solids.

Supplemental fuel is needed for heating up an incinerator until it reaches combustion temperatures. Supplemental fuel may be required during routine operation, depending on the overall heat balance for the incinerator. Normally, either natural gas (37,300 kJ/m^3 or 1,000 Btu/ft^3) or heavy fuel oils ($3.9x10^7$ kJ/m^3 or $1.05x10^6$ Btu/ft^3 for #2 grades and $4.0x10^7$ kJ/m^3 or $1.07x10^6$ Btu/ft^3 for #4 grades) are selected. In the case of incineration of anaerobically digested sludge, the ability to use recovered excess digester gas (about half the thermal value of natural gas) is an option for supplemental fuel.

Incinerator ash should be disposed of in a properly designed ash pond or approved landfill.

Sludges should ideally be concentrated to a solids concentration where they will burn autogenously. This solids concentration will vary somewhat with sludge type, volatile solids percentage and the chemical composition of the solids, but a concentration in the order of 30 percent TS will generally be required. A detailed mass and energy balance should be provided by the designer to calculate energy requirements and allowable solids loading rate. The design calculations should define the potential for heat recovery from the exhaust gases based on the overall mass and energy balance calculations.

Two types of incinerators are generally used for wastewater sludge: multiple-hearth and fluidized bed. Fluidized Bed Incineration (FBI) is considered the type of incineration which is the state-of-the-art system and is usually the type that is provided for a new installation. The multiple hearth incinerator technology has largely been replaced by the FBI for sludge incineration.

16.5.6.1 Fluidized Bed Incinerator

The Fluidized Bed Incinerator (FBI) is a vertical, cylindrical, refractory-lined steel shell that contains no internal moving mechanical parts, only a sand bed and fluidizing air diffusers. It is normally available in sizes from 2.7 to 7.6 m (9 to 25 ft) in diameter. Units with diameters up to 10.7 m (35 ft) have been built. The sand bed is approximately 0.76 m (2.5 ft) thick at rest and sits on a refractory-lined grid. This grid contains *tuyeres* through which air is injected into the furnace at a gauge pressure of 20 to 34 kPa (3 to 5 psi) to fluidize the bed, which expands to approximately 200 percent of its at-rest volume.

Sludge is fed directly into the fluidized sand bed. Due to the fluidization, there is violent mixing in the bed which provides rapid and uniform distribution of fuel and air and consequently, good heat transfer and combustion. Heat transfer and reaction between the gases and the solids are rapid because of the large surface area available. The bed provides substantial heat storage capacity, which helps to reduce short-term temperature fluctuations that may result from varying feed sludge heating values or moisture contents. Organic particles remain in the sandbed until they are reduced to mineral ash. The violent motion of the bed comminutes the ash material, preventing the buildup of clinkers (agglomerated material). The resulting fine ash is stripped from the bed by the upflowing gases and carried out the top of the furnace and removed by air pollution control devices.

The temperature in the bed is maintained between 760 and 815°C (1,400 and 1,500°F) by addition, if needed, of supplemental fuel directly into the bed. For short periods of time during start up of the fluidized bed, an auxiliary burner(s) located either above or below the sand bed is utilized. In installations with autogenous combustion, a water spray (cooling system) above the bed is used to regulate the furnace temperature. In essence, the reactor is a single chamber unit in which both drying and combustion occur in the sandbed. All of the combustion gases pass through the combustion zone, with residence times of several seconds at 760 to 815°C (1,400 and 1,500°F).

The required air flow into the furnace is determined by several factors. Fluidizing and combustion air should be sufficient to expand the bed to a proper density, yet low enough to prevent the sludge from rising to and floating on top of the bed. Too much air can blow sand and products of incomplete combustion into the off-gases, while too little air can cause oxygen levels to fall below stoichiometric requirements for complete oxidation of all volatile solids in the sludge feed. Once the air flow rate has been determined in design, only minor adjustments are required during operation. Temperatures should be sufficiently high to assure complete deodorizing, but low enough to protect the refractory, heat exchanger and flue gas ducting. Optimum thermal economy is generally obtained with 20 to 45 percent excess air, a figure which varies as a function of the feed sludge composition.

The combustion process should be followed by an air emission control system to meet the requirements of *Air Pollution-Local Air Quality*, (O. Reg. 419/05), made under the *Environmental Protection Act*. The designer should refer to *Section 3.11 - Emissions of Contaminants to Air*.

16.6 ODOUR CONTROL

Odour control facilities should be provided for sludge mixing and treated sludge storage tanks when located within 0.8 km (0.5 miles) of residential or commercial areas. The designer should refer to *Section 4.4 - Odour Control and Abatement Measures*.

Ventilation is required for indoor sludge mixing, storage or processing facilities (*Section 7.2.10 - Safety Ventilation*). Ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least 12 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Air should be forced into the area by mechanical means rather than solely exhausted from the area. The air change requirements should be based on 100 percent fresh air. Portable ventilation equipment should be provided if there is no permanently installed ventilation equipment.

Anaerobic digester pressure relief valves should be designed and checked to prevent unnecessary release of digester gas. Similarly, compressor systems should be designed, checked and maintained to prevent gas leakage that could be either a safety problem or odour source.

16.7 SLUDGE PUMPS AND PIPING

16.7.1 Sludge Pumps

Pump capacities should be adequate but not excessive. Provision for varying pump capacity is desirable. Duplicate units, or sufficient units with at least the largest unit out-of-service, should be provided at all installations.

Pumps with demonstrated solids handling capability should be provided for handling raw and digested sludge. Where centrifugal pumps are used, a parallel positive displacement pump may be provided as an alternate to pump heavy sludge concentrations, such as primary or thickened sludge, that may exceed the pumping head of the centrifugal pump. Some pumps require upstream grinders for successful operation.

A minimum positive head of 610 mm (24 in) should be provided at the suction side of centrifugal pumps and is desirable for all types of sludge pumps. Maximum suction lifts should not exceed 3 m (10 ft) for plunger pumps. Unless sludge sampling facilities are otherwise provided, quick-closing sampling valves or piston valves should be installed at the sludge pumps. The size of valve and piping should be at least 40 mm (1.5 in) and terminate at a suitable location.

16.7.2 Sludge Piping

Digested sludge withdrawal piping should have a minimum diameter of 200 mm (NPS-8) for gravity withdrawal and 150 mm (NPS-6) for pump suction and discharge lines. Where withdrawal is by gravity, the available head on the discharge pipe should be at least 1.2 m (4 ft) and preferably more. Undigested sludge withdrawal piping should be sized in accordance with *Section 11.3.2.3 - Sludge Removal Pipeline*.

Gravity piping should be laid on uniform grade and alignment. Slopes on gravity discharge piping should not be less than 3 percent for primary sludges and all sludges thickened to greater than 2 percent solids. Slopes on gravity discharge piping should not be less than 2 percent for aerobically digested sludge or waste activated sludge with less than 2 percent solids. Where gravity sludge transfer is proposed, provision should be made for a pumped transfer on a regular basis to remove deposits and clean out the lines. The pumped operation may be necessary after a few years of gravity operation when the gravity operation is not possible due to sludge deposits. Valving should be provided to allow for both gravity and pumped transfer.

Cleanouts should be provided for all gravity sludge piping. Provisions should be made for draining and flushing discharge lines. All sludge piping should be suitably located or otherwise adequately protected to prevent freezing. The section of piping between isolation valves should have a drain and vent valves or other means to relieve built-up pressure, due to gas formation, prior to dismantling the piping for cleaning or repairs.

Special consideration should be given to the corrosion resistance and permanence of supporting systems for piping located inside the digestion tank.

CHAPTER 17

SLUDGE THICKENING AND DEWATERING

CHAPTER 17

SLUDGE THICKENING AND DEWATERING

This chapter describes dewatering of solids (sludge and biosolids) prior to disposal or prior to further treatment and stabilization. This includes sludge/biosolids conditioning, gravity and mechanical thickening, mechanical dewatering and drying beds for dewatering.

A summary of the thickening and dewatering performance expectations for conventional processes is provided in Appendix V, which should be used in conjunction with the details in this chapter.

17.1 GENERAL

The sludge solids concentrations that are listed in *Section 16.1.1 - Sludge Quantities and Characteristics* and Table 16-1 are the concentrations which can generally be achieved without the use of separate thickening or dewatering facilities. To achieve any significantly higher concentrations, sludge thickening and/or dewatering facilities will be required. The designer needs to evaluate the dewatering requirements for any planned sludge utilization/disposal operation and the end-use.

Sludge thickening normally refers to the process of reducing the free water content of sludges; whereas, dewatering refers to the reduction of floc-bound and capillary water content of sludges. See Tables 17-1 and 17-2 for typical performance expectations for various thickening and dewatering processes, respectively.

The benefits which can be derived from reductions in sludge water content include:

- Reduction in digester sizing requirements to achieve the same solids retention time;
- Reduction in heat exchange capacity requirements for anaerobic digestion;
- Reduction in sludge pumpage and transportation costs;
- Decrease in ultimate disposal costs;
- Reduction of handling problems and leachate production during sludge landfilling operations;
- Sizing for subsequent treatment or stabilization options [e.g. sludge drying, autothermal aerobic digestion (ATAD)]; and
- Reduction in storage volume is required to comply with the *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management*

Act. For STPs which are not phased in under the *Nutrient Management Act*, requirements are set out in the Certificate of Approval (C of A), based on the MOE and the Ministry of Agriculture, Food and Rural Affairs' *Guidelines for the Utilization of Biosolids and Other Wastes on Agricultural Land, 1996*.

There may be some disadvantages to excessive reduction in water content which should also be taken into consideration:

- Sludge mixing and blending facilities may be required to combine sludges of differing water content for subsequent treatment operations;
- Sludge at 12 to 15 percent total solids (TS) is not free flowing and may require special sludge handling equipment;
- Dewatered sludges because of their significant loss in plant available nitrogen content may not be as acceptable or desirable for spreading on agricultural lands as liquid sludges are; and
- Contaminant loading in recycle streams requiring treatment.

Wherever possible, pilot plant and/or bench-scale data is recommended for the design of sludge thickening and dewatering facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters should be used. The following subsections outline the normal ranges for the design parameters of such equipment.

In considering the need for sludge thickening and dewatering facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

Recycle streams (i.e., centrate and filtrate) need to be carefully considered and their impact on the liquid train of the sewage treatment plant (STP) accounted for. The impact of these recycle streams need to be related to their schedule of operation, as thickening and dewatering processes are often operated only during the day shift and on weekdays. With or without equalization, the loading of these streams will increase the loading to the liquid train. These recycle streams may be small in terms of their volumetric loading, but can be concentrated in terms of organic loading, especially total ammonia-nitrogen loading from dewatering of biosolids from anaerobic digestion processes. Also the operation of the thickening and dewatering processes should be coordinated with the operation of the sludge feed processes, as these processes may be continuous and the thickening/dewatering processes may be operated periodically (e.g. 7 to 8 hours per day, 5 days per week). The impact should be assessed based on the actual flow and biological load from the dewatering facility on the liquid train performance, calculated over the actual time when the sidestream is returned rather than averaged over a 24-hour period, unless equalization of the recycle flows is provided.

Instrumentation should be considered for all thickening and dewatering processes to monitor influent and effluent flows, and sampling should confirm influent, thickened or cake and recycle stream solids. Additional sampling of the recycle stream is recommended to confirm the loading to the liquid train.

17.2 SLUDGE CONDITIONING

Sludge thickening and sludge dewatering operations (depending on the process used), are highly dependent upon sludge conditioning for their effective operation. Sludge conditioning affects the solids concentration of the thickened or dewatered sludge and the solids capture efficiency.

There are several characteristics of sludges, including industrial-derived constituents which may adversely affect attempts to achieve solid-liquid separation. The presence of colloidal particles increases the specific resistance of the sludge and adversely affects sedimentation processes. The net negative charge exhibited by most sewage sludges tends to make the particles repulse each other and thus resist agglomeration into larger particles. Sludge particles have a bound water content which, if retained, results in low cake solids after solid-liquid separation. Sludge conditioning operations attempt to alter one or more of the above sludge characteristics so as to improve the efficiency of the solid-liquid separation processes.

There are two sludge conditioning approaches that can be used. Sludge can be conditioned by physical methods, such as heat treatment or addition of fly ash or by chemical methods, involving the addition of either coagulants and/or polymers.

The method selected will not only differ in its effect on the thickening or dewatering process, but will have different effects on subsequent sludge handling operations and on the STP itself, due to recycle streams.

17.2.1 Chemical Methods

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. Chemical conditioning for thickening operations attempts to promote more rapid phase separation, higher solids concentration and a greater degree of solids capture. With dewatering operations, chemical conditioning is used in an attempt to enhance the degree of solids capture by destabilization and agglomeration of fine particles. This promotes the formation of a cake, which then becomes the true filter media in the dewatering process.

With most thickening operations and with belt filter press dewatering operations, the most commonly used conditioning chemicals are polymers. For dewatering by vacuum filtration, ferric salts, often in conjunction with lime, are most commonly used. Chemical conditioning using polymers is most prevalent with centrifuge dewatering, with metal salts being avoided mainly due to corrosion problems. For dewatering by filter presses, the use of high molecular weight polymers for sludge conditioning has been successfully employed in lieu of lime and ferric chloride. The ultimate disposal methods may also have an effect on the choice of conditioning chemicals. For instance, lime and ferric compounds should be avoided with incineration options.

The selection of the most suitable chemical(s) and the appropriate dosage requirements for sludge conditioning can best be determined by pilot and full-scale testing and optimization. Pilot- or full-scale testing should assess the impact of residual polymer on the liquid train processes and consider the additives increase in metal content of the sludge.

Laboratory testing should be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15 percent of full-scale needs.

Mixing of the chemicals may be accomplished by either diffused air or mechanical mixers. If diffused aeration is used, an air supply of 0.85 L/(m^3·s) (51 cfm/1000 ft^3) of mixing tank volume should be provided. When diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service. If mechanical mixers are used, the impellers need to be designed to minimize fouling with debris in the sludge and consideration should be made to provide continuity of service during freezing conditions.

17.2.2 Physical Methods

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. Heat conditioning can be accomplished by either a non-oxidative or oxidative system. With this process, the sludge is treated at temperatures of 175 to 204°C (347 to 399ºF), pressures of 1,700 to 2,800 kPa (247 to 406 psi) and for detention times of 15 to 40 minutes. The high temperatures cause hydrolysis of the encapsulated water solids matrix and lysing of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the solid-liquid separation characteristics.

This process may result in a significant organic loading to the biological treatment process of the sewage treatment plant, if the supernatant is returned to the bioreactor, due to the solubilization of organic matter during sludge hydrolysis. This liquor can represent 25 to 50 percent of the total loading on the secondary treatment process and allowances should be made in the STP design to accommodate this loading increase.

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion resistant materials such as stainless steel. Scale formation in the heat exchangers, pipes and reactor will require acid washing equipment to be provided.

Heat conditioning, particularly the non-oxidative process, can also result in the production of odorous gases. If ultimate sludge disposal is via incineration, these gases can be incinerated in the upper portion of the furnace [760°C (1,400ºF) or higher]. If incineration is not a part of the sludge handling process, a catalytic or other type of oxidation unit should be used.

The design requirements for a heat conditioning system should be determined by either batch or small-scale continuous pilot-plants. Through such methods, the necessary level of hydrolysis to produce the desired reduction in the specific resistance of the sludge and the liquor characteristics can be determined. Tests can also be made at different temperatures and detention times to determine the most effective full-scale operating conditions.

Another common form of physical conditioning is the addition of admixtures such as fly ash, incinerator ash, diatomaceous earth or waste paper. These conditioning techniques are most commonly used with filter presses or vacuum filters. The admixtures when added in sufficient quantities produce a porous lattice structure in the sludge which results in decreased compressibility and improved filtering characteristics. When considering such conditioning techniques, the beneficial and detrimental effects of the admixture on such parameters as overall sludge mass, calorific value, should be evaluated along with the effects on improved solids content.

Although once widely use as a conditioning technique, elutriation is no longer a popular process and is not be covered in these guidelines.

Freezing of sludges has been used successfully in Ontario for water treatment plant sludges, but there are no known systems intentionally using slow freezing as a conditioning method for STP sludges. Thawed sludge releases its moisture more rapidly than sludge that has not been frozen and the sludge is left in a light, fluffy condition. The process reportedly produces good results for subsequent gravity dewatering of the thawed sludge (up to 16 percent solids for waste activated sludge (WAS) and up to 25 percent solids for digested sludge). The disadvantages of the system are the high BOD_5 of the effluent and the high cost of the process unless natural means of freezing can be used.

17.3 SLUDGE THICKENING

Sludge thickening can be employed in the following locations in an STP:

- Prior to digestion for raw primary, secondary sludge or mixed primary and secondary sludges;
- Prior to dewatering facilities;
- Following digestion for sludges or supernatant; and
- Following dewatering facilities for concentration of filtrate, decant, or centrate.

The commonly used methods of sludge thickening and their suitability for the various types of sludge are shown in Table 17.1. In selecting a design figure for the thickened sludge concentration, the designer should recognize that thickening devices are adversely affected by high sludge volume index (SVI) and benefit by low SVI in the activated sludge. The ranges of thickened sludge concentrations given in the table below assume an SVI of approximately 100 mL/g. Thickening targets should also consider digestion needs. Pre-thickening sludge to greater than 4 percent TS prior to aerobic digestion can lead to autothermal digestion and issues associated with this process such as odours and foaming problems.

17.3.1 Design Considerations

Sludge thickeners to reduce the volume of sludge should be considered to reduce the required digester *capacity*. The design of thickeners (gravity, dissolved-air flotation, centrifuge, gravity belt thickeners, rotary drum screens and others) should consider the type and concentration of sludge, the downstream sludge stabilization processes, dewatering and storage requirements, the method of ultimate sludge disposal, chemical needs and the cost of operation.

Particular attention should be given to the pumping and piping of the concentrated sludge and possible onset of anaerobic conditions and impact of corrosion. Provision should be made for draining and flushing of discharge lines.

The designer should consider odour and aerosol/humidity control for all thickening technologies. Wherever thickening devices are being installed, special consideration should be given to the need for sludge pretreatment in the form of sludge grinding to avoid plugging pumps, lines and thickening equipment. Also, where thickeners are to be housed, adequate ventilation and odour control will be required, meeting all applicable codes.

17.3.2 Gravity Thickening

Gravity thickening is principally used for primary sludge and mixtures of primary and waste activated sludges. Due to the better performance of other thickening methods for WAS, gravity thickening has limited application for such sludges.

Gravity thickeners should be designed in accordance with the following parameters:

Tank Dimensions

- Tank shape - circular;
- Tank sidewater depth - 3 to 3.7 m (9.8 to 12.1 ft);
- Tank diameter - up to 21 to 24 m (69 to 79 ft); and
- Floor slope - acceptable range of 2:12 to 3:12.

Solids Loadings

- Primary sludge - 96 to 120 kg/(m^2·d) [20 to 25 lb/(ft^2·d)];
- Waste activated sludge - 12 to 36 kg/(m^2·d) [2.5 to 7.4 lb/(ft^2·d)];
- Combination of primary and waste activated sludges based on weighted average of above loading rates; and
- Use of metal salts for phosphorus removal will increase solids loading rates by at least the stoichiometric amount.

Overflow Rate

- To prevent septic conditions, an overflow rate of 0.19 to 0.38 L/(m^2·s) (0.28 to 0.56 USgpm/ft^2) is recommended.

Mechanical Rake

- Rake should have a tip speed of 50 to 100 mm/s (9.8 to 19.7 ft/min);
- To be equipped with hinged lift mechanisms when handling heavy sludge such as lime-treated primary sludge, otherwise optional; and
- Surface skimmer is recommended.

Sludge Underflow Piping

- Keep length of suction lines as short as possible; and
- Dual sludge withdrawal lines should be considered.

Chemical Conditioning

- Provision should be made for the addition of conditioning chemicals into the sludge influent lines (polymers, ferric chloride or lime are the most likely chemicals to be used to improve solids capture).

17.3.3 Dissolved Air Flotation

Unlike heavy sludges, such as primary and mixtures of primary and waste bioreactor sludges, which are generally most effectively thickened in gravity thickeners, light waste bioreactor sludges can be successfully thickened by dissolved air flotation (DAF).

The advantages of DAF compared with gravity thickeners for excess secondary treatment sludges include its reliability, production of higher sludge concentrations and better solids capture. Its disadvantages include the need for greater operating skill and higher operating costs.

Experience has shown that DAF operations cannot be designed on the basis of purely mathematical computations or by the use of generalized design parameters. Some bench- and/or pilot-scale testing will be necessary. The following, design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operation:

Tank Dimensions

- Air buoyancy systems - vary with suppliers;
- Air-to-solids weight ratio - 0.02 to 0.05; and
- Recycle ratios - vary with suppliers (0 to 500 percent).

Solids Loadings (With WAS to achieve 4 percent Float Solids)

- 48 kg/(m^2·d) [10 lb/(ft^2·d)] (without flocculating chemicals); and
- Up to 240 kg/(m^2·d) [49 lb/ (ft^2·d)] (with flocculating chemicals).

Chemical Conditioning

- Feed chemical to mixing zone of sludge and recycled flow;
- Most installations now use chemical conditioning with polymers to achieve more economical operation; and
- Polymer feed range 0 to 25 g/kg (0 to 50 lb/ton) of dry solids.

Hydraulic Feed

- Up to 1.74 L/(m^2·s) (2.56 USgpm/ft^2) (based on total flow including recycle, when polymers used);
- Without chemicals, lower rate should be used; and
- Feed rate should be continuous rather than intermittent.

Detention Time

- Not critical providing that particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

Thickened Sludge Withdrawal

- Surface skimmer moves thickened sludge over dewatering beach into sludge hopper;
- Either positive displacement or centrifugal pumps that will not air bind should be used to transfer sludge from hopper to the next phase of process; and
- In selecting pumps, maximum possible sludge concentrations should be taken into consideration.

Bottom Sludge

- A bottom collector to move settled sludge into a hopper should be provided; and
- Sludge removal from the hopper may be by gravity or pumping.

17.3.4 Centrifugation

Centrifuges are commonly used for sludge dewatering and are increasingly being considered for sludge thickening. As thickening devices, their use has been generally restricted to waste activated sludges. Three types of centrifuges have been used with such sludges - the solid-bowl decanter, disc-nozzle and basket types.

The following general design considerations are provided:

- Centrifugal thickening operations can have substantial maintenance and operating costs;
- Where space limitations or sludge characteristics make other methods unsuitable or where high capacity mobile units are needed, centrifuges have been used;
- Thickening capacity, thickened sludge concentration and solids capture of a centrifuge is greatly dependent on the SVI of the sludge;
- 85 to 95 percent solids recovery will generally be the most suitable operating range;
- Polymer feed range 0 to 6.0 g/kg (0 to 128.0 lb/ton) of dry solids;
- Early experience with disc nozzle-type centrifuges found clogging of the sludge discharge nozzles to require frequent maintenance; recent use of rotary screens and cyclones for pretreatment have helped alleviate these problems; and
- Basket type centrifuges have seen limited use; due to their low capacities and batch operations, their use has been generally restricted to small plants.

17.3.5 Gravity Belt Thickener

The gravity belt thickener (GBT) uses a slow moving fabric belt to separate sludge solids and free water. Polymer is required to precondition the sludge and is prepared and aged in a small tank upstream of the thickening process. Sludge thickening on the device is aided by multiple rows of plows and drainage elements which slow the flow of sludge and provide additional retention time over the horizontal gravity belt. GBTs generally require a smaller footprint than other sludge thickening processes, are cost-effective and use less energy than other mechanical thickening devices (i.e., DAF and centrifuge). However, GBTs require preconditioning with chemicals and are sensitive to the quality of the sludge being thickened.

The following are general design considerations:

- Performance of the GBTs is subject to upstream conditions in the STP. The better the settling of solids in the plant, the better the GBT will function and potentially at lower chemical dosages;
- Adequate attention should be given to transporting the thickened solids, in particular for handling the maximum solids content expected;
- Prior to digestion, adequate mixing or blending of thickened solids with other solids is required;

- Plows on the gravity belt turn and distribute the thickened solids to allow for water to drain through the belt fabric. The number and location should be adjustable for each type of sludge being thickened;
- Chemical addition and mixing equipment are important, as are multiple injection points;
- GBTs should have an air handling system to maintain a safe working environment; this could include a complete enclosure with exhaust, odour control, inspection door, and access for cleaning;
- GBTs should have a curb around them and floors sloped to drains so that operators can properly clean the equipment;
- Metering of solids into and out of the equipment is important;
- Thickened solids need to be designed to move all expected material and avoid accumulation and overload;
- Due to height of equipment, an elevated walkway will probably be needed to operate and maintain the equipment; and
- Scum (grease) should not be placed on the GBT because blinding of the fabric can create problems.

17.3.6 Rotary Drum Thickener

Rotary drum screen thickeners are internally fed with sludge from a headbox or flocculation tank after conditioning with polymer. The suspension is distributed onto the internal surface of the rotating drum and physically strained for the separation of free water. The cylinder can be fitted with interchangeable screening panels and is slowly rotated (e.g. 2-10 rpm) with a variable speed drive electric gear motor. Separated solids are retained on the surface of the screen and are conveyed to the discharge end of the unit where they drop out through a chute. The rotary drum thickener (RDT) process has a built-in spray backwashing system, controlled with programmable timers that can be optimized for each application. The rotational speed of the drum can be adjusted and optimized based on site-specific operating requirements to achieve the desired levels of thickened sludge concentration, solids capture efficiency and polymer consumption. The system can be supplied as an enclosed unit with a vent stack for containment and minimization of odour and vapour releases.

As with GBTs, chemical addition is required. Similar design considerations to the GBT should be considered, although as this process is enclosed, odour and environmental issues are reduced.

Table 17- 1 - Sludge Thickening Methods and Performance with Various Sludge Types

Thickening Method	Sludge Type	Expected Performance
Centrifugation	Waste Activated with Polymer[1]	8-10% TS and 80-90% Solids Capture with Basket Centrifuges; 4-6% TS and 80-90% Solids Capture with Disc-nozzle Centrifuges; 5-8% TS and 70-90% Solids Capture with Solid Bowl Centrifuges.
Gravity Belt Thickener (GBT)	Waste Activated with Polymer	4-8% TS and ≥ 95% Solids Capture
Rotary Drum Thickener (RDT)	Waste Activated with Polymer	4-8% TS and ≥ 95% Solids Capture
Gravity	Raw Primary	8-10% TS
	Raw Primary and Waste Activated	5-8% TS
	Waste Activated	2-3% TS (Better results reported for oxygen rich activated sludge)
	Digested Primary Digested Primary and Waste Activated	8-14% TS
Dissolved Air Flotation (DAF)	Waste Activated (Not Generally Used for Other Sludge Types)	4-6% TS and ≥ 95% Solids Capture With Flotation Aids

Note:
1. Solids concentrations for centrifuges without polymer will be reduced.

17.4 SLUDGE DEWATERING

Sludge dewatering will often be required at STPs prior to ultimate disposal of sludge/biosolids or as a prelude to further treatment or stabilization. Since dewatering processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major influence on the dewatering method most suitable for a particular STP. Also of influence will be the characteristics of the sludge requiring dewatering; that is, whether the sludge is raw or digested, whether the sludge contains WAS or whether the sludge has been previously thickened. With raw sludge, the freshness of the sludge will have a significant effect on dewatering performance (septic sludge will be more difficult to dewater than fresh raw sludge).

A rational basis of design for sludge production values should be developed. In lieu of actual sludge production data, an overall plant mass balance should be provided to account for sludge production from each treatment unit process, including the liquid and solids trains and the influence of recycle streams on the main liquid train treatment processes.

Table 17-2 gives the solids capture, solids concentrations normally achieved, energy requirements and suitable ultimate disposal options for various dewatering methods. The solids concentrations shown in the table assume that the sludges have been properly conditioned. Designers should be aware that phosphorus removal chemicals (i.e., alum or ferric chloride) will reduce allowable solids loading rates for dewatering equipment and produce a lower cake solids concentration than would be expected without phosphorus removal. This is expected due to the additional sludge loading due to the chemical sludge and its potential lower initial concentration.

Table 17- 2 - Sludge Dewatering Methods and Performance with Various Sludge Types

Dewatering Method	Solids Capture (%)	Solids Concentrations Typically Acheived[1]	Median Energy Required (MJ/dry t)[2] [Btu/lb]
Belt Filter Press	85-95	Raw or Digested Primary + WAS (14-25%) WAS (10-15%)	130 [67]
Centrifuge (Solid Bowl)	95-99	Raw or Digested Primary + WAS (15-30%) WAS (12-15%, with polymer)	360 [155]
Filter Press	90-95	Raw Primary + WAS (30-50%) Digested Primary + WAS (35-50%) WAS (25-50%)	360 [155]
Vacuum Filter	90-95	Raw Primary + WAS (10-25%) Digested Primary + WAS (15-20%) WAS (8-12%)	1080 [464]

Notes:

1. Including conditioning chemicals (i.e., polymer).
2. MJ/dry tonne - denotes megajoules per dry tonne of sludge throughput.

The required solids concentration for sludges which are to be landfilled at municipal sanitary landfill sites are normally specified by the landfill authority. With small quantities of sludge for co-disposal land filling with garbage, liquid sludge at solids concentrations as low as 3 percent TS may be acceptable. For sludge-only landfill operations, a minimum of 15 percent solids concentration is generally required to support cover material. If sludge is to be disposed of in sludge lagoons, dewatering may not be necessary unless it is justifiable for economic reasons relating to haulage costs.

For ultimate disposal by incineration, sludges should ideally be concentrated to a solids concentration where they will burn autogenously or will be self-generating. This solids concentration will vary somewhat with sludge type, volatile solids percentage and the chemical composition of the solids, but a minimum concentration in the order of 30 percent total solids will generally be required. With conditioning by heat treatment, sludge dewatering methods such as filter presses, belt filter presses, centrifuges and perhaps even vacuum filters will be capable of producing autogenous sludge solids concentrations.

As with thickening systems, dewatering facilities may require sludge pretreatment in the form of sludge grinding to avoid plugging pumps and lines and plugging or damaging dewatering equipment. Also, adequate ventilation equipment and odour control will be required in buildings housing dewatering equipment, meeting all applicable codes.

In evaluating dewatering system alternatives, the designer should consider the capital and operating costs, including labour, parts, chemicals and energy, for each alternative as well as for the effects that each alternative will have on the sewage treatment and subsequent sludge handling and ultimate disposal operations. Since labour and especially energy costs are escalating at a rapid rate, it is suggested that these annual costs be converted to capital cost equivalents for evaluation purposes (i.e., life-cycle costing approach should be used).

17.4.1 Mechanical Dewatering Facilities

Provision should be made to maintain sufficient continuity of service so that sludge or biosolids may be dewatered without accumulation beyond storage capacity. The number of vacuum filters, centrifuges, filter presses, belt filters, other mechanical dewatering facilities or combinations thereof should be sufficient to dewater the sludge produced with the largest unit out of service (i.e., firm capacity). Unless other standby wet sludge facilities are available, adequate storage facilities of at least 4-days production volume prior to dewatering in addition to any other sludge storage needs should be provided.

Back-up vacuum and filtrate pumps should be provided. Overall, the design of dewatering facilities should facilitate the removal and replacement of all equipment.

Adequate facilities should be provided for ventilation of the dewatering area. The exhaust air should be properly conditioned to avoid odour nuisance.

Lime-mixing facilities should be completely enclosed to prevent the escape of lime dust. Chemical handling equipment should be automated to eliminate manual lifting requirements.

17.4.2 Centrifuges

The centrifuge types which have been used for sewage sludge dewatering include the solid bowl, basket and disc centrifuges. The most frequently used is the continuous countercurrent solid bowl centrifuge. Due to their infrequent use for dewatering and their inherent plugging problems, disc centrifuges, although capable of reaching the lower end of dewatering solids concentrations, are generally only used for thickening operations and will not be discussed further in this dewatering section.

17.4.2.1 Solid Bowl Centrifuges

The designer should consult the manufacturer to obtain the equipment design parameters.

The machine variables of importance for dewatering centrifuges include bowl length/diameter ratio, bowl angle, bowl flow pattern, bowl speed, pool volume, internal conveyor design and relative conveyor speed.

Bowl length/diameter ratios of 2.5 to 4.0 are usually provided to ensure adequate settling time and surface area. Bowl angles should be kept shallow.

The bowl flow pattern can be either countercurrent or co-current. By using co-current flow, the settled sludge is not disturbed by the incoming feed and turbulence is reduced. The disadvantages of co-current flow are the need for a long feed tube and the long travel distance needed to remove the sludge. Other proprietary feed inlets have also been developed to minimize the disturbance to the previously settled solids.

Increased bowl speed increases the centrifugal forces available for clarification, but the settled solids become more difficult to remove due to the higher gravitational (G) forces. Increased bowl speed, however, will also increase abrasion damage within the centrifuge, noise and vibration. Lower speed machines have been developed to achieve high solids capture. Sludge inlet conditions with these low-speed machines have also been improved to minimize acceleration and turbulence. These low-speed machines have lower noise levels, minimized internal wear and have lower power requirements, but may have increased conditioning chemical requirements.

Detention time in the centrifuge will increase with increases in pool volume. With longer detention times achieved by greater pool depth, solids capture increases, but cake solids concentrations will decrease due to reduced detention time on the drying deck and also due to the capture of finer solids with higher moisture content. Pool depth can be varied by adjustable weirs.

Conveyor design and speed will affect the efficiency of solids removal. Differential speed should be kept low enough to minimize turbulence and internal wear yet high enough to provide sufficient solids handling capacity. The most suitable internal conveyor pitch will be affected by the characteristics of the sludge to be handled. With high solids concentration, conveyors with high pitch angles can be used, but with lower solids, low pitch angles should be used. Conveyor differential speed can often be optimized following installation.

Important process variables affecting the centrifuge efficiency are feed rate, feed consistency, temperature and the chemical coagulants used.

As hydraulic flow rate increases through a centrifuge, solids capture decreases and cake solids concentrations increase due to the loss of fines in the centrate. If the feed solids are too high, solids buildup within the bowl can take place, reducing clarification volume. Therefore, both solids and hydraulic overloading can occur.

For most sewage sludges, the capacity of the centrifuge will be limited by the clarification capacity (hydraulic capacity) and therefore the solids concentration. Increasing the feed solids will increase the solids handling capacity. Thickening should, therefore, be considered as a pretreatment operation.

Since temperature affects the viscosity of sludges, if temperatures vary appreciably (as with aerobic digestion), the required centrifuge capacity should be determined for the lowest temperature expected.

The chemical conditioning agents most commonly used with centrifuges are polymers. Flocculating agents are generally injected directly into the interior of the centrifuge to avoid shearing the floc. Maximum effectiveness is generally achieved by diluting the flocculant to concentrations of 0.1 percent or less.

Other general design guidelines for solid bowl centrifuges are as follows:

- Feed pump - sludge feed should be continuous; pumps should be variable flow type; one pump should be provided per centrifuge for multiple centrifuge systems; chemical dosage should vary with pumpage rate;
- Return line – should be included for wet cake during start-up;
- Sludge pretreatment - depending upon the sewage treatment process, grit removal, screening or maceration may be required for the feed sludge stream;
- Solids capture - 85 to 95 percent generally desirable;
- Machine materials - generally carbon steel or stainless steel; parts subject to wear should be protected with hard facing materials such as a tungsten carbide material;
- Machine foundations - foundations should be capable of absorbing the vibratory loads; and
- Provision for maintenance - sufficient space should be provided around the machine(s) to permit disassembly; an overhead hoist should be provided; hot and cold water supplies will be needed to permit

flushing out of the machine; drainage facilities will be necessary to handle wash water.

17.4.3 Belt Filter Presses

Although variations in the process exist, a belt filter press (BFP) basically consists of two continuous, separate belts: a press belt and a filter belt. Sludge is confined between the two belts with the press belt exerting pressure on the filter belt, thereby continuously dewatering the sludge.

There are generally three distinct dewatering zones through the process. The first zone is a gravity drainage zone, the second is a pressure zone and the third is a shear zone. Pressure is exerted by the rollers, conveying belts or other external devices. In the shear zone, the sludge cake is further dewatered by deforming the sludge cake by passing the belts around rolls and/or between vertically offset rollers causing a serpentine configuration in the sludge cake movement.

Most types of sewage sludges can be dewatered with BFPs and the results achieved are generally superior to those with vacuum filters. Belt filter presses generally use only one-third the power requirements of vacuum filters and do not experience sludge pickup problems often encountered with vacuum filters. BFPs have reportedly been used to further dewater the sludge cake from vacuum filters with excellent results. Such a method should be considered for upgrading existing dewatering systems. Chemical conditioning is generally with polymers.

Solids handling capabilities are likely to range from 50 g/($m^2 \cdot s$) [0.61 lb/(ft^2.min)] of dry solids (based on belt width) for waste activated sludge to 330 g/($m^2 \cdot s$) [4.1 lb/(ft^2.min)] for primary sludge. Expected solids concentration results are shown in Table 17-2. Solids capture is usually in excess of 85 percent and often as high as 95 percent.

An enclosure should be considered for each BFP to contain odours and splashing, including air exhaust with odour control.

17.4.4 Vacuum Filters

Rotary vacuum filters were commonly used mechanical systems for sludge dewatering in Ontario. With the recent developments of new, lower cost and more effective dewatering systems the use of vacuum filters is beginning to decline.

Rotary drum, rotary belt and spring coil variations of the rotary vacuum filter are available for use. The primary machine variables which affect dewatering are vacuum pressure, drum submergence, drum speed, degree of sludge agitation and filter medium. The operation variables which affect dewatering performance are sludge type, sludge conditioning and sludge characteristics including initial solids concentration, nature of sludge solids, chemical composition, sludge compressibility, sludge age, temperature and filtrate viscosity.

Of primary importance with vacuum filters is the solids concentration of sludge feed. With all other operating variables remaining constant, increases in filtration rates vary in direct proportion to feed solids. Sludge thickening prior to vacuum filters is therefore extremely important. Higher concentrations in the sludge feed also result in lower filtrate solids.

Vacuum filtration systems should be designed in accordance with the following parameters:

- Sludge feed pumps - variable capacity;
- Vacuum pumps - generally one per machine with a capacity of 10 L/(m^2·s) (15 USgpm/ft^2) at 65 kPa (9.4 psi) vacuum or more;
- Vacuum receiver - generally one per machine; maximum air velocity 0.8 to 1.5 m/s (2.6 to 4.9 ft/s); air retention time 2 to 3 minutes; filtrate retention time 4 to 5 minutes; all lines to slope downward to receiver from vacuum filter;
- Filtrate pumps - generally self-priming centrifugal; suction capacity greater than vacuum pump [65 to 85 kPa vacuum (9.4 to 12.3 psi)]; with flooded pump suctions; with check valve on discharge side to minimize air leakage into the system; pumps should be sized for the maximum expected sludge drainage rates (usually produced by polymers);
- Sludge flocculation tank - constructed of corrosion-resistant materials; with slow speed variable drive mixer, detention time 2 to 4 minutes with ferric and lime; with polymers, shorter time may be used;
- Wash water - filtered final effluent generally used;
- Sludge measurement - should be provided unless measured elsewhere in plant; and
- Solids loading rate - 7-14 g/(m^2·s) [0.086 - 0.17 lb/(ft^2.min)] for raw primary; 2.8-7 g/(m^2·s) [0.034 - 0.86 lb/(ft^2.min)] for raw primary + WAS; 4-7 g/(m^2·s) [0.049 - 0.86 lb/(ft^2.min)] for digested primary + WAS; not considered practical for use with WAS alone.

17.4.5 Filter Presses

Recent changes in the design of filter presses, including elimination of leakage problems, more automation, improved filter media, greater unit capacities and development of high molecular weight polymers and compatible polymer feed systems, have resulted in renewed interest in this method of sludge dewatering.

Variations in the filter press process which are now available on the market include units with recessed plates or plates with frames, top or central sludge feed, air pressure assisted sludge cake release, automatic washing of filter media, sequential or simultaneous release of sludge cake and final compression stage using flexible diaphragm behind filter media.

The main advantage offered by filter presses is the ability to concentrate all types of sewage sludges to very high concentrations. Concentrations as high as 45 to 50 percent TS, can generally be achieved with properly conditioned sludges. Filter presses are also able to effect high efficiencies in solids capture and as a result produce relatively clear filtrate. Their primary disadvantages are that the process is batch rather than continuous and cake removal still requires some manual assistance and large quantities of conditioning agents are generally necessary.

As with vacuum filters, the capacity of filter presses are greatly affected by the initial solids concentration. With lower feed solids, chemical requirements increase significantly.

Sludge thickening should therefore be considered as a pretreatment step. The sludge is generally conditioned with a physical conditioning agent such as fly ash or with chemicals such as ferric chloride, lime and alum, although use of polymer conditioning agents is becoming more common with the development of compatible polymer feed systems. In some instances, precoats are applied to the filter media prior to the addition of sludges to prevent premature media blinding. Various materials can be used for precoat including diatomaceous earth, fly ash, incinerator ash and various types of industrial waste by-products.

Filter press systems should be designed in accordance with the following guidelines:

- Sludge conditioning tank - detention time maximum of 20 minutes at peak pumpage rate;
- Feed pumps - variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge; pumps should be of a type to minimize floc shear; pumps should deliver high volume at low head initially and low volume at high head during latter part of cycle; ram or piston pumps, progressing cavity pumps or double diaphragm pumps are generally used;
- Cake handling - filter press should be elevated above cake conveyance system to allow free fall; cake can be discharged directly to trucks, into dumper boxes or onto conveyors (usually belt or drag chain type); conveyors should be able to withstand impact of sludge cakes; cable cake breakers may be needed;
- Cycle times - 1.5 to 6.0 h; and
- Operating Pressures - usually 700 to 1400 kPa (102 to 203 psi), but may be as high as 1750 kPa (254 psi).

Operating pressures depend on the types of presses and the chemical agents used for sludge conditioning. These pressures may be developed either hydraulically or by a combination of hydraulic and pneumatic means. For example, recessed plate filter presses with diaphragm membranes for dewatering polymer-conditioned sludges are first brought to approximately 700 kPa (102 psi) pressure hydraulically (pumping) and then the membranes are inflated pneumatically to provide a final squeezing pressure of approximately 1050 kPa (152 psi).

While the magnitude of pressure applied does not adversely affect the dewatering process, if lime and ferric chloride are used as sludge conditioning, it is very important that the generating pressure should not exceed 1050 kPa (152 psi) if polymer is applied as the conditioning agent, due to the interaction of the conditioners.

17.4.6 Filtrate and Drainage/Centrate Management

Filtrate or drainage from sludge drying beds and centrate from other dewatering units should be returned to the liquid train of the STP at appropriate points and rates. Appropriate monitoring and sampling of these streams should be provided.

17.4.7 Other Dewatering Processes

If it is proposed to dewater sludge by other alternative or innovative methods, a detailed description of the process and design data (including field or pilot data) should accompany the design report.

17.5 SLUDGE DRYING BEDS

Sludge drying beds may be used for dewatering stabilized sludge from either the anaerobic or aerobic process. Drying beds are confined, underdrained and shallow layers of sand over gravel on which wet sludge is distributed for draining and air drying. Drying beds have proved satisfactory at most small- and medium-sized sewage treatment plants located in warm, dry climates.

Sludge drying bed area should be calculated based on:

- The volume of wet sludge produced by existing and proposed processes; and
- The time required on the bed to produce a removable cake. Adequate provision should be made for sludge dewatering and/or sludge disposal facilities for those periods of time during which outside drying of sludge on beds is hindered by weather.

Owing to simple operation, capability of producing high solids concentrations (greater than 40 percent TS) and low capital cost, conventional sand drying beds should be considered as a sludge dewatering alternative, especially for small to medium-sized STPs. Due to the presence of sludge drying beds at a large number of existing Ontario STPs, their use as an emergency sludge dewatering technique to backup mechanical dewatering processes should also be considered.

Since sludge conditioning can reduce the required drying time to one-third or less of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

The usual design parameters for conventional sand drying beds are as follows:

- Drainage tile 100 mm (4 in) diameter or more, spaced 2.4 to 3.0 m (8 to 10 ft) apart, with slope of one percent or more;
- Bottom of cell should be of impervious material such as clay or asphalt;
- Drainage tile bedded in gravel layer usually 200 mm to 500 mm (8 to 20 in) thick, graded from 25 mm (1.0 in) on the bottom to 3 mm (0.12 in) on the top;
- Sand layer above gravel usually 250 to 450 mm (10 to 18 in) thick with an effective size of 0.3 to 1.2 mm (0.012 to 0.047 in) and a uniformity coefficient of less than 5.0;
- Bed size 4.5 to 7.5 m (15 to 25 ft) wide with length selected to satisfy desired bed loading volume;
- Dosing depth 200 to 300 mm (8 to 12 in) for warm weather operating modes; for winter freeze drying cumulative depths of 1 to 3 m (3.3 to 10.0 ft) can be used depending upon the number of degree days in winter;
- One inlet pipe per cell, with inlet 300 mm (11.8 in) above bed surface and with splash pad to prevent bed disruption and to promote even distribution of sludge; provision for flushing inlet lines should be provided;
- Usually a minimum of 3 beds are desirable for flexibility of operation;
- Sludge removal can either be manual or mechanical; if mechanical, concrete vehicle tracks are generally required with clay tiles, but may not be necessary with perforated plastic pipe and/or flotation tire equipped front end loader;
- Outer walls and partition walls should be watertight; walls should extend 460 mm (18 in) above and at least 230 mm (9 in) below the surface of the bed. Outer walls should be watertight down to the bottom of the bed and extend at least 100 mm (4 in) above the outside grade elevation to prevent soil from washing into the beds;
- Each bed should be constructed so as to be readily and completely accessible to mechanical equipment for cleaning and sand replacement. Concrete runways spaced to accommodate mechanical equipment should be provided. Special attention should be given to

assure adequate access to the areas adjacent to the sidewalls. Entrance ramps down to the level of the sand bed should be provided. These ramps should be high enough to eliminate the need for an entrance end wall for the sludge bed;

- Underdrains should discharge back to the secondary treatment section of the STP; and
- Recommended sizing for uncovered beds between latitudes 40 to 45°N is 0.16 m^2/cap (1.72 ft^2/cap) and for north of 45°N, 0.20 m^2/cap (2.15 ft^2/cap) (for primary plus waste activated sludge following anaerobic digestion); the recommended sizing for covered beds with the same sludge type is 0.13 and 0.16 m^2/cap (1.40 and 1.72 ft^2/cap), respectively. Equivalent mass loading rates can be calculated using Table 16-1; and
- Consideration should be given for providing a means of decanting the supernatant of sludge placed on the sludge drying beds. More effective decanting of supernatant may be accomplished with polymer treatment of sludge.

Other types of sludge drying beds that have been used include the following:

- Paved rectangular beds with a centre sand drainage strip, with or without heating and with or without covering;
- "Wedge-wire" drying beds with a wedge-wire system; provision for an initial flood with a water layer, followed by sludge introduction on top of water layer, controlled cake formation and provision for controlled underdrainage and mechanical sludge removal; and
- Rectangular vacuum assisted sand beds.

17.6 SLUDGE THICKENING LAGOONS

Thickening lagoons have generally been built at or near the site of the STPs so that the sludge can be conveyed to the lagoons by pumping or gravity and so that the supernatant can be returned to the STP for further treatment.

In some circumstances where suitable land surrounds the lagoon, a combination thickening and transfer lagoon can be built where supernatant can be spray irrigated onto the surrounding land and the thickened sludge can then be hauled away for spreading on farmland. Withdrawal of supernatant will result in increases in sludge concentration to the extent that sludge removal by pumping may become difficult or impossible; above a solids concentration of 7 to 8 percent TS, pumping can become difficult.

The design and location of sludge thickening lagoons should take into consideration many factors, including the following:

- Possible nuisances - odours, appearance, mosquitoes (*Chapter 4 - Odour Control)*;

- Design - number, size, shape and depth;
- Loading factors - solids concentration of digested sludge, loading rates;
- Soil conditions - permeability of soil, need for liner and stability of berm slopes;
- *Groundwater* conditions - elevation of maximum groundwater level, direction of groundwater movement, location of wells in the area;
- Sludge and supernatant removal - volumes, concentrations, methods of removal, method of supernatant treatment and final sludge disposal; and
- Climatic effects - evaporation, rainfall, freezing, snowfall, temperature, solar radiation.

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CHAPTER 18

SLUDGE STORAGE AND DISPOSAL

CHAPTER 18

SLUDGE STORAGE AND DISPOSAL

Sludge produced by treatment of sewage may require temporary storage prior to processing, disposal or utilization. This chapter provides guidance for designing sludge or biosolids storage systems. Disposal options, including land application and landfilling, are also described.

18.1 GENERAL

Sludge storage facilities should be provided for all mechanical sewage treatment plants. Storage facilities may consist of any combination of drying beds, lagoons, separate tanks and pad areas to store liquid, dewatered or dried sludge. Additional volume in biosolids stabilization units may be used, but this is usually limited in capacity. The design should provide for odour control in sludge storage tanks and sludge lagoons. Refer to *Chapter 17 - Sludge Thickening and Dewatering* for information on sludge volume reduction and drying.

Calculations to establish the number of days and volume of storage should be carried out and should be based on the total sludge handling and disposal system. Refer to *Chapter 16 - Sludge Stabilization* for sludge characteristics and generation rates of various sewage treatment processes.

The *General Regulation* (O. Reg. 267/03) made under the *Nutrient Management Act* should be consulted for storage capacity requirements for biosolids destined to agricultural land application. If the land application method of sludge disposal is the only means of disposal used at a sewage treatment plant (STP), storage should be provided in accordance with O. Reg. 267/03 based on considerations of the following items:

- Inclement weather effects on access to the application land;
- Temperatures including frozen ground and stored sludge cake condition, (land application of biosolids during the period beginning on December 1 of one year and ending on March 31 of the following year or at any other time when the soil is snow-covered or frozen is not allowed by O. Reg. 267/03);
- Haul road restrictions including spring thawing conditions;
- Seasonal rainfall patterns of the area;
- Cropping practices on available land;
- Potential for increased sludge volumes from industrial sources during the design life of the plant;
- Available area for expanding sludge storage; and
- Pathogen reduction and vector attraction reduction requirements.

A minimum of 240 days storage should be provided for the ultimate design *capacity* of the STP unless a different period is justified on the basis of the site specific conditions. The minimum 240 days storage requirements under O. Reg. 267/03 can be a combination of a permanent biosolids nutrient storage facility, a temporary field nutrient storage site (dewatered municipal sewage biosolids only) or a combination of such facilities and sites that is capable of storing generated sewage biosolids during a period of at least 240 days.

18.2 SLUDGE STORAGE LAGOONS

Fully treated (i.e., stabilized) sludge may be stored in lagoons prior to further processing or disposal. Planning and design should address design elements such as dike engineering and liners and consider factors including long-term planning issues.

Sludge should be stabilized in such a way that offensive odours do not result from a lagoon site. Where sludge lagoons are used, adequate provisions should be made for alternative acceptable sludge handling methods in the event of upset or failure of the sludge stabilization process.

Sludge lagoons should be located as far as practicable from inhabited areas or areas likely to be inhabited during the lifetime of the structures.

Adequate provisions should be made to seal the sludge lagoon bottoms and embankments to prevent leaching into adjacent soils or *groundwater*. The seal should be protected to prevent damage from sludge removal activities. Exfiltration of water from the lagoon may not be acceptable and, if occurring, should meet the requirements of the *ministry* Guideline B-7, *Incorporation of the Reasonable Use Concept into Groundwater Management* which provides the framework for determining acceptable off-property impacts on groundwater resources. The designer is referred to *Section 12.3.4 - Lagoon Construction* for additional information.

Provisions should be made for pumping or heavy equipment access for sludge removal from the sludge lagoon on a routine basis.

Lagoon supernatant should be returned to the STP at locations and rates that minimize the effect on the treatment processes.

Two types of sludge lagoon systems have been most commonly constructed in Ontario - thickening lagoons and sludge transfer site lagoons.

18.2.1 Thickening Lagoons

Thickening lagoons have generally been built at or near the site of the sewage treatment plants so that the sludge can be conveyed to the lagoons by pumping or gravity flow and that supernatant can be returned to the STP for further treatment.

18.2.2 Sludge Transfer Site Lagoons

Sludge transfer site storage lagoons have usually been built as temporary sludge storage facilities. These sites may be required as part of a program for biosolids utilization on agricultural land to hold biosolids during times of the year when land spreading cannot be carried out. With true transfer site lagoons, no particular attempt is usually made to withdraw supernatant or to otherwise thicken the sludge beyond the natural thickening that occurs due to evaporation. Where suitable land surrounds the lagoon, a combination of thickening and transfer lagoon may be used where supernatant may be spray irrigated onto the surrounding land and the thickened biosolids can be hauled away for spreading on farmland. Withdrawal of supernatant may result in increases in sludge concentration to the extent that sludge removal by pumping may become difficult or impossible, particularly above a total solids (TS) concentration of 7 to 8 percent.

These lagoons require the ministry approval in accordance with Ontario Regulation. 347, *General – Waste Management,* made under the *Environmental Protection Act.*

The designer should refer to the ministry document "*Guide on Applying for Approval of Waste Disposal Sites*" for additional information.

18.2.3 Anaerobically Digested Sludge Storage

Anaerobically digested solids may be stored in covered basins or facultative solids basins. The anaerobically digested solids storage facility should be designed to abate vector attraction and odour conditions. The facultative solids storage basin should be designed to maintain an aerobic surface layer free of scum accumulation. The organic loading rate for a facultative solids storage basin should not exceed 0.1 kg VS/m^2 (20 lb/100 ft^2) of surface area per day. Surface aerators should be used to maintain the aerobic zone and break up surface film. The surface aerators should be designed to minimize the mixing action between the aerobic and anaerobic zones. The facultative solids basin should have a minimum sidewater depth of 3.7 m (12 ft). The top 0.9 m (3 ft) should be kept aerobic.

18.2.4 Aerobically Digested Sludge Storage

Aerobically digested solids may be stored for extended periods of time and the basin contents should be kept thoroughly mixed using diffused air or mechanical aeration. A minimum air requirement of 30 m^3/(1000 m^3·min) (30 cfm/1000 ft^3) should be provided. If mechanical surface aerators are used, a minimum power requirement of 2.6 kW/1000 m^3 (0.1 hp/1000 ft^3) should be provided.

18.2.5 Alkaline Stabilized Sludge Storage

Liquid alkaline stabilized high pH (>12) sludge should not be stored in a lagoon, but should be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or re-treatment. Provisions should be made for adding alkaline material in the storage tank. Mixing equipment should be provided in all storage tanks.

On-site storage of dewatered alkaline stabilized sludge should be limited to 30 days. Provisions for rapid re-treatment or disposal of dewatered sludge stored on-site should be made in case of sludge pH reduction.

18.3 STORAGE FOR SLUDGE OR BIOSOLIDS

18.3.1 Dewatered Sludge or Biosolids

Dewatered sludge with a solids content of less than 35 percent may be stored on-site up to 7 days. An excess capacity should be provided due to inclement weather or other factors that do not allow transport or disposal. The dewatered sludge may be stored in steel or concrete containers and should be located and stored to preclude re-wetting by rainfall.

Dewatered sludge with solids content greater than or equal to 35 percent may be stored on-site for up to 90 days. The dewatered sludge may be stored in containers or in stockpiles. The storage facility should be located to preclude groundwater contamination. Open stockpiles should include provisions for collecting rainfall runoff. All rainfall runoff should be collected and returned to the head of a treatment facility.

18.3.2 Dried Sludge and Biosolids

Dried sludge with a solids content of greater than or equal to 50 percent may be stored on-site in bins or covered facilities. Enclosed structures may produce explosive gaseous byproducts or dust. The enclosed area of the storage structure should be sufficiently ventilated to eliminate the accumulation of dangerous gas mixtures. The enclosed storage structure should be mechanically ventilated with approximately 20 to 30 air changes per hour. All exhaust air should pass through an odour control system.

18.3.3 Sludge Storage Tanks and Basins Prior to Dewatering

Holding tanks and basins are commonly provided as an integral part of most conditioning processes and many stabilization processes. Tanks and basins may be used for blending materials such as sewage sludge from primary and secondary clarifiers.

Large storage tanks are generally constructed of concrete. Smaller tanks are often constructed of carbon steel with a suitable coating or liner. Tank and basin equipment often includes an aeration system, mechanical mixers or a recycling system for mixing. All equipment within the tank should be constructed of a corrosion-resistant material such as polyvinyl chloride (PVC), polyethylene (PE), stainless or glass-lined steel.

Tanks and basins may be sized to retain still-to-be dewatered sewage solids (liquid sludge/biosolids) for a period of several hours to a few days. If sewage solids are stored longer than two or three days, the product may deteriorate and can become difficult to dewater.

If the tank or basin is a closed vessel, the designer should ensure that there are access portholes for inspection and maintenance. All access portholes need to meet the Canadian Gas Association (2005) *Code for Digester Gas and Landfill Gas Installation*, CAN/CGA-B105-M93, 1993 requirements.

Short storage periods of unstabilized primary and secondary sewage sludges in a holding tank or basin can produce nuisance odours. Decanting tanks following thermal conditioning can often create odour problems. The design should include assessment of odour potential and provide for sufficient odour control equipment to minimize odour emissions.

18.3.4 Bulk Storage

Design considerations for a bulk storage area are as follows:

- The size of the biosolids storage area depends on the quantity of biosolids produced, when it can be used and its moisture content;
- Drier solids can be stacked higher with less tendency to slump;
- Additional space should be provided for scheduled process cleaning (lagoon dewatering or digester cleaning) and emergency situations; and
- Materials should not be stored in a manner that will likely result in contamination of ground or *surface waters*, air or land in case of flood or fire.

The storage area should be constructed and sited to prevent run-on and runoff of liquids. A solids storage area needs a water collection system and a way to treat the leachate produced from the pile. Care should be taken not to contaminate the solids with oil, grease, gas, rocks and litter. The area needs to be secure to prevent access by the public, domestic animals or wildlife.

Depending on the population proximity and density of the area, quality of the biosolids (stability) and prevailing winds, odour control should be provided to minimize the impact on the surrounding neighborhood. In addition, mechanical ventilation should be provided for enclosed storage sites.

18.4 LAND APPLICATION

Biosolids may be used as a soil conditioner for agricultural, horticultural or reclamation purposes depending on the degree of stabilization provided. The quantity of solids generated by the selected treatment process should be calculated or estimated from similar full-scale facilities or pilot facilities. Pathogen levels and concentrations of metals in sludge should be determined using standard laboratory test procedures. Pathogen levels and concentration of metals should be less than the levels specified in O. Reg. 267/03 if land application is used. A mass balance approach should be used to determine the quantity of biosolids produced at the facility.

Important design considerations include but are not necessarily limited to:

- Type of sludge/biosolids stabilization process;
- Pathogen and vector attraction reduction to levels specified in O. Reg. 267/03;
- Biosolids characteristics including the presence of inorganic and organic chemicals;
- Application site characteristics (e.g. soils, groundwater elevations, setback distance requirements);
- Local topography and hydrology;
- Type of crop and land to which biosolids can be applied in accordance with O. Reg. 267/03. For STPs which are not phased in under the *Nutrient Management Act*, requirements are set out in the Certificate of Approval (C of A), based on the MOE and the Ministry of Agriculture, Food and Rural Affairs' *Guidelines for the Utilization of Biosolids and Other Wastes on Agricultural Land, 1996*.
- Cropping practices, spreading and incorporation techniques;
- Population density and odour control; and
- Sampling, health and safety requirements in accordance with O. Reg. 267/03.

A contingency plan should be provided for flexibility in the event of equipment failure or conditions that prevent the primary use or disposal method. The design should account for weather factors such as rainfall, wind conditions and humidity in the selection of the use or disposal of sewage sludge. Due to inclement weather and cropping practices, alternative storage or disposal options are recommended to ensure the biosolids are properly managed. Mixing equipment or provisions to assist in the monitoring of land-applied biosolids should be considered in the design of biosolids handling and storage facilities.

Municipal sewage treatment plants that apply sewage biosolids on agricultural land are required under O. Reg. 267/03 to have prepared a nutrient management strategy (NMS) and have the strategy approved by the Ontario Ministry of Agriculture, Food and Rural Affairs (OMAFRA).

18.5 DISPOSAL

Immediate sludge disposal may be used to reduce the sludge inventory at the STP site and amount of sludge that may need to be retreated to prevent odours if sludge pH decay occurs.

18.5.1 Landfilling

Sludge may be disposed of in approved municipal sanitary landfills. The required solids concentration for sludges that are to be landfilled at sanitary landfill sites are normally specified by the landfill authority. With small quantities of sludge for co-disposal landfilling with municipal solid waste, liquid sludge at solids concentrations as low as 3 percent (TS) may be acceptable. For sludge-only landfill operations, a minimum of 18 percent TS concentration or more, or a slump of 150 mm (6 in) or less, is generally required to support cover material. The "*Test Method for Determination of Liquid Waste (Slump Test)*" is set out in Schedule 5 of Ontario Regulation 347 *General – Waste Management*, made under the *Environmental Protection Act*.

CHAPTER 19

CO-TREATMENT OF SEPTAGE AND LANDFILL LEACHATE AT SEWAGE TREATMENT PLANTS

CHAPTER 19

CO-TREATMENT OF SEPTAGE AND LANDFILL LEACHATE AT SEWAGE TREATMENT PLANTS

This chapter includes design considerations for treatment of septage, landfill leachate and other wastes hauled to municipal sewage treatment plants. Introducing these hauled waste loadings into *sewage works* for co-treatment with sewage exerts demands on the processes that are disproportionate to typical sewage hydraulic and organic loadings. Smaller sewage treatment plants in particular should be aware, before accepting such loadings, that a single load may overload and potentially upset their processes and may cause effluent quality deterioration.

19.1 SEPTAGE

19.1.1 General

One method of septage disposal and treatment is discharge to a municipal sewage treatment plant (STP).

The term “septage” for the purpose of these guidelines refers to the contents removed from septic tanks, portable toilets, privy vaults and holding tanks serving houses, schools, motels, mobile home parks, campgrounds and small commercial endeavors, all receiving sewage from domestic sources. Although septic tank waste may be the highest in concentration of this group and potentially more stabilized than other holding tank type wastes, these wastes are generally combined in the load discharged at the plant and therefore they are often considered as a group. In cases where the hauled waste is solely holding tank waste, the characteristics would be similar to regular domestic sewage.

19.1.2 Characteristics

Compared to raw domestic sewage from a conventional municipal sewage collection system, septage usually has high concentrations of organics, grease, nutrients, hair, stringy material, scum, grit, solids and other extraneous debris. Substantial quantities of phosphorus, Total Kjeldahl Nitrogen (TKN), total ammonia-nitrogen (TAN), bacterial growth inhibitors and cleaning materials may be present in septage depending on the source, typical concentrations are provided in Table 19-1. A comparison of some of the common parameters for septage and municipal sewage is given in Table 19-2.

Data for local septage to be received should be collected for design of septage receiving and treatment facilities. The characteristics of septage should be expected to vary widely from load to load depending on the source. If co-treated at an STP, treatment of septage will increase the amount of solids generated.

Table 19-1 - Physical and Chemical Characteristics of Septage

Parameter	Concentration (mg/L)			EPA Mean[1]	Suggested Design Value[1, 2]
	Avg.	Min.	Max.		
TS	34,100	1,100	130,500	38,800	40,000
TVS	23,100	400	71,400	25,300	25,000
TSS	12,900	300	93,400	13,000	15,000
VSS	9,000	100	51,500	8,700	10,000
BOD_5	6,500	400	78,600	5,000	7,000
COD	31,900	1,500	703,000	42,800	15,000
TKN	600	100	1,100	700	700
TAN	100	5	120	160	150
Total P	200	20	760	250	250
Alkalinity	1000	500	4,200	-	1,000
Grease	5,600	200	23,400	9,100	8,000
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.

Table 19- 2 - Comparison of Contaminant Concentrations in Septage and Sewage

Parameter	Septage[1] (mg/L)	Sewage (mg/L)	Ratio of Septage to Sewage
TS	40,000	720	55:1
TVS	25,000	360	69:1
TSS	15,000	210	71:1
VSS	10,000	160	62:1
BOD_5	7,000	190	37:1
COD	15,000	430	35:1
TKN	700	40	17:1
TAN	150	25	6:1
Total P	250	7	36:1
Alkalinity	1,000	90	11:1
Grease	8,000	90	89:1
pH	6.0	-	-
Linear Alkyl Sulphonates (LAS)	150	-	-

Notes:

1. Suggested design values (see Table 19-1).

The general characteristics for septage that may be hauled to an STP are discussed below. The designer should evaluate the potential impacts of these characteristics and assess whether the plant has enough *capacity* and proper equipment to co-treat septage effectively considering the large variability in the physical-chemical characteristics of septage. Due to the contents and nature of solids in septage, their impact on existing grit removal equipment, pumps and air diffusers should be specifically evaluated.

In many respects, septage is similar to domestic sewage, except that septage is significantly more concentrated. Differences in contaminant concentrations between septage and domestic sewage are outlined below:

BOD_5 - The BOD_5 of septage can be as much as 30 to 50 times or more concentrated than that of normal domestic sewage.

TSS - Compared to domestic sewage, septage can be very high in total suspended solids (e.g. 10 to 50 times typical STP influent total suspended solids). Evaluation of solids characteristics of local septage waste is recommended and should include total solids (TS), total suspended solids (TSS), total volatile solids (TVS) and settleable solids (SS).

Fats, Oils and Grease - Almost no decomposition of grease occurs at a sewage treatment works and the expense of handling and disposing grease can be considerable. Contents from grease traps should not be hauled to municipal STPs for treatment. Rendering and other recycling options are often available and preferable to handling such matter at a sewage treatment works.

Grit - A household septic tank will accumulate grit, rocks and other dense material in its sediment layer over the years. After cleaning out many septic tanks, the accumulation of this sediment load in the septage hauling tank can be several hundred kilograms. Because of this concern for downstream sedimentation, discharge into a sewage collection system should be avoided. The septage receiving station at the STP site should have provisions for an adequate rock sump. Even with an adequate rock sump, dense grit can form a compacted layer in a sewer after several years of routine septage discharge into the collection system.

Odour - Due to the anaerobic nature of a septic tank system and the mixture of organic materials, septage is very odorous. Design should include means to control these potential sources of odour.

Nutrients - The concentrations of nitrogen and phosphorus in septage are high compared to typical domestic sewage and needs to be accounted for. This is especially important if nitrification and/or biological nutrient removal is required at the STP.

Heavy Metals - Metals in septage may come from household chemicals, leaching of plumbing pipes and fixtures and possible contamination from previous industrial loads hauled in the septage hauling truck. Because metals do not decompose and the interval between septic tank pumpings can be several years, metals tend to accumulate in septage.

19.1.3 Treatment

Septage is normally considered treatable at STPs when proper engineering design is provided. The designer should consider the following factors to minimize shock loadings or other adverse impacts on plant processes and effluent quality:

- Hydraulic capacity [m^3/d (mUSgd)] of the plant relative to the volume and rate of septage directed to the plant;
- Unused plant capacity available (above current sewage collection system loadings) to treat septage loadings;
- Sensitivity of the treatment plant process to daily fluctuations in loadings brought about by the addition of septage (e.g. slug loads);
- Slug septage loadings of BOD_5, TKN, TAN or phosphorus which may cause process upset (e.g. to the nitrification process, if applicable), odour nuisance, aeration tank/aerated digester foaming or pass through to the STP effluent;
- The point of introduction of the septage into the STP process. Feasible alternative points of feed to the treatment units should be evaluated including feed to the sludge processing units provided the unit function will not be adversely affected. Generally septage can be introduced at the following locations within a plant:
  - Upstream of the primary clarifier;
  - Directly into an anaerobic digester but with caution not to overload the digester [maximum Total Volatile Solids (TVS) loading should be controlled];
  - At small extended aeration plants, septage can be added to an aerobic digester;
- Blending septage with sludge prior to dewatering should not be considered;
- The ability to control feed rates of septage to the plant for off-peak loading periods;
- The volume and concentrations of bacterial growth inhibitors in septage from some portable vault toilets and recreational dump station holding tanks; and
- Treatment of septage in a sewage stabilization pond (lagoon) can be done but requires evaluation depending on the number of stages (lagoons in series) and the amount of septage received compared to sewage flow and may require equalization of septage. TAN removal may be a problem due to high loads from the septage and additional

treatment such as intermittent sand filtration may be required following lagoon treatment. (*Section 12.3.6 – Intermittent Sand Filters*)

The effluent quality requirements on each of the controlled parameters should be considered when evaluating these factors.

If the septage is primarily septic tank solids, which are partially stabilized, this material may be successfully introduced into the sludge treatment process (i.e., digesters) after pretreatment. Loadings need to be carefully reviewed, potential impacts on sludge treatment processes (e.g. dewatering) assessed and equalization considered. Since septage in Ontario is often a combination of septic tank and holding tank wastes, it is generally first treated through the STP liquid train.

19.1.4 Design Criteria

The decision to treat septage flows as a part of the conventional municipal sewage treatment process has several significant effects. Treating these flows increases the load on both the liquid and solids trains of the STP with resulting increases in operating cost, solids production, solids handling and disposal or utilization costs. Accepting this loading consumes a greater proportion of the capacity than similar volumes of normal sanitary sewage flow. Treating septage flows can affect the ongoing operation and the quantity of biosolids produced at a given facility.

Water Environment Federation (WEF) *Septage Handling - Manual of Practice No. 24* (1997) and other references provide ranges of design values. Although literature values for BOD_5 and other septage constituent concentrations are available, assessment of the actual septage composition and volumes that are expected locally should provide the basis for design.

Design of the STP processes should account for septage loading as a part of the complete design. Loading assumptions and design criteria for septage receiving should be calculated separately in addition to the main influent sewage loading assumptions.

19.1.5 Considerations

In the case of existing STPs, it is essential that an adequate engineering evaluation be made of the existing processes and the anticipated septage loading prior to receiving septage at the plant. For proposed plant expansion and upgrading, the engineering report needs to include anticipated septage loading in addressing treatment plant sizing and process selection (*Chapter 2 - Project Design Documentation*). The following items should be included as appropriate in the engineering evaluation:

- The continuous and satisfactory treatment (i.e., continuously meeting the plant effluent quality criteria) of sewage loads from the sewer system should not be adversely affected by the addition of septage to the plant for co-treatment;

- The smaller the plant design capacity relative to the septage loading, the more susceptible the plant will be to process upset and potential violation of effluent quality requirements;
- Allocation of plant organics treatment capacity originally planned for future growth;
- For plants to be expanded and upgraded, the engineering evaluation should include the sensitivity of the treatment process(es) to receiving septage and the impact on the effluent quality. Amendments need to be made to the *Certificate of Approval* of the sewage works to account for new septage receiving facilities;
- An evaluation of available plant operator staff and the staffing requirements necessary when septage is to be received;
- The space for constructing septage receiving facilities that is to be off-line from the raw sewage incoming from the sewer system. The location of the septage receiving facility and the septage hauler unloading area should consider other plant activity and traffic flow;
- The amount of additional sludge generated will depend on the type of treatment used, the septage-to-sewage ratio and septage characteristics; and
- The impact of the septage handling and treatment on the plant sludge handling and processing units and ultimate sludge disposal procedures.

Several software packages are now available for simulating models of sewage treatment processes and full-plant models. Based on a calibrated model of the existing facility under current operating conditions, estimates of the effects of increased organic or hydraulic loadings (e.g. associated with septage additions) can be made.

19.1.6 Receiving Facility

The design of the septage receiving station at the sewage treatment plant should provide for the following elements:

- A hard surface haul truck unloading ramp sloped to a drain to allow ready cleaning of any spillage and washing of the haul tank, connector hoses and fittings. The ramp drainage should be tributary to treatment facilities and should exclude excessive stormwater;
- A flexible hose fitted with easy connect coupling to provide for direct connection from the haul truck outlet to minimize spillage and help control odours;
- Electronic metering and billing systems are available to monitor septage received and provide accurate billing information to septage haulers and plant staff. These systems generally consist of a card reader or key pad for controlled access in combination with a flow meter and valve;

- Washdown water with ample pressure, hose and spray nozzle for convenient cleaning of the septage receiving station and haul trucks. The use of chlorinated effluent may be considered for this purpose;
- An adequate off-line septage receiving tank should be provided. The tank should be sized to hold twice the maximum daily volume of septage expected on a peak day. Capability to collect a representative sample of any truck load of waste accepted for co-treatment at the plant should be provided. The receiving tank should be designed to provide complete draining and cleaning by means of a sloped bottom equipped with a drain sump. The design should give consideration to adequate mixing. Adequate mixing will ensure uniformity of septage strength and mixing for chemical addition, if necessary, for treatability and odour control;
- Screening, grit and grease removal or grinding of the septage as appropriate to protect the STP process units;
- Pumps provided for handling the septage should be of the non-clogging design and capable of passing 100 mm (4 in) diameter solids;
- Glass-lined pipes are recommended;
- Valving and piping for operational flexibility to allow the control of the flow rate and point of septage discharge to the STP; and
- Safety features to protect the operational personnel.

19.2 LANDFILL LEACHATE

Landfill leachate is produced as a result of rain percolating through the landfill waste and reacting with the products of decomposition, chemicals and other materials in the waste. Typically, landfill leachate is anoxic, acidic, rich in organic acid groups, TKN, TAN, sulphate and chloride ions and with high concentrations of common metal ions especially iron.

Acceptance of leachate for co-treatment at an STP should be assessed and then carried out with the same precautions outlined in the guidelines provided above for septage. Due to the extreme variability of leachate, dependent on its source, it should be reviewed on a case-by-case basis to determine its properties. Unlike septage, leachate can contain components of industrial nature and a more complex sampling program may be necessary. Potential inhibitory effects of leachate constituents and loadings on STP processes (e.g. on nitrification, if applicable) need to be assessed.

Pretreatment of landfill leachate should be considered to reduce the strength and equalize the loading to the STP. A variety of biological and chemical treatment options can be used to pretreat leachate on-site. For biological treatment, characterization and treatability tests should be conducted to ensure the leachate on its own is treatable without additional nutrients and chemical adjustments.

19.3 OTHER TYPES OF HAULED WASTE

Other wastes may be hauled to the sewage treatment plant, but these like all septage and hauled wastes from industrial and commercial sources should be characterized and expected loads and delivery times known. The capacity and capability of the STP to co-treat these waste types need to be assessed on a case-by-case basis. This will ensure that the impact of these wastes on the STP operations can be determined before being accepted. It is inappropriate for hazardous and flammable wastes to be co-treated at STPs and are excluded.

19.3.1 Chemical Toilet Waste

Materials from portable toilet facilities are commonly called chemical toilet waste. Portable toilets are pumped similarly to septic tanks and transported to a treatment works for co-treatment. Commonly a chemical is added to the portable toilet's holding tank to control odours. Characteristics of chemical toilet waste need to be assessed for potential impacts on the STP.

19.3.2 Recreational Vehicle Waste

The characteristics of recreational vehicle (RV) waste are similar to chemical toilet waste.

19.3.3 Vactor Waste

Many sanitary sewer collection systems use vacuum maintenance equipment to clean sewer lines, catchbasins, manholes and pump station wet wells. Depending upon the source, the resulting composition of the vactor load can vary widely. A full vactor truck may contain materials from several different types of cleaning assignments. Any vactor spoils contaminated with sewage should be properly treated and disposed. If vactor wastes are received from sources other than sanitary sewers, these wastes need to be characterized before being accepted.

19.3.4 Waste from Other Sewage Treatment Plants

Sludge or sewage received from other sewage treatment facilities should be assessed on a case-by-case basis.

19.3.5 Water from Soil Remediation

Water byproduct from soil remediation processes (mostly *groundwater*) should be assessed before discharging to municipal sewage treatment plants.

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CHAPTER 20

HANDLING OF CHEMICALS

CHAPTER 20

HANDLING OF CHEMICALS

This chapter outlines the handling of chemicals that are commonly used at sewage treatment plants (e.g. for phosphorus removal, disinfection, and sludge thickening and dewatering).

20.1 GENERAL

Chemicals selected for use in sewage treatment plants (STPs) should not adversely affect the operation of the sewage or sludge treatment processes and should not leave dangerous or harmful residuals in the effluent or sludge leaving the plant. The purity of chemicals proposed to be used should be determined. Occasionally, waste streams from industry, such as ferrous chemicals, can be used at STPs provided that they are not contaminated with other hazardous materials.

Laboratory, pilot- or full-scale studies of various chemicals, feed points and applicable treatment processes are recommended for STPs to determine the achievable performance level, cost-effective design criteria and ranges of chemical dosage requirements.

The selection of appropriate chemicals and dosages for STPs should be based on such factors as influent sewage characteristics, the proposed chemical, effluent quality requirements and applicable treatment process requirements.

Common uses of chemicals and hazardous materials at sewage treatment facilities include sewage treatment, process enhancement and control, housekeeping, landscaping, laboratory, maintenance, fuel supply and odour control. Common chemicals and hazardous materials used in STPs include, but are not limited to, those shown Table 20-1.

20.2 STORAGE OF CHEMICALS

Storage should be provided for at least thirty days of consumption at the maximum anticipated chemical usage rate, allowing for variations in sewage flow and chemical dosage in that period. Where deliveries of chemicals may be interrupted by adverse weather conditions in isolated locations, provision should be made for increased storage capacity taking into consideration that some chemicals degrade with time (e.g. sodium hypochlorite solution). Where deliveries can be assured on short notice and the material is not essential, storage requirements can be reduced.

Table 20-1 - Common Chemicals and Hazardous Chemicals Used in STPs

Treatment Chemical	Combustible, Hazardous, Flammables and Explosive Hazards
Alum	Activated carbon
Ammonia	Acetylene
Caustic (e.g. sodium hydroxide)	Diesel fuel
Chlorine	Digester gas
Chlorine dioxide	Fuel oil
Defoamers	Gasoline
Ferric chloride	Liquefied petroleum (LP) gas (propane)
Ferric sulphate	Lubricating oils
Ferrous sulphate	Methanol
Ferrous chloride	Natural gas
Hydrochloric acid	Paints and thinners
Hydrogen peroxide	Pesticides
Lime	Solvents
Odour-masking agents	Welding gases
Oxygen	
Ozone	
Polymers	
Sodium bisulphite	
Sodium hypochlorite	
Sodium thiosulphate	
Sulphur dioxide	
Sulphuric acid	

Some of the considerations that will have an effect on the chemical storage volume requirements are as follows:

- Chemical usage rate and delivery time;
- Typical volume delivered (storage should be at least one truck load plus 25 percent);
- Availability of alternate suppliers;
- Loss in strength of chemical with storage time;
- Seasonal changes in chemical strength; and
- Critical nature of chemical with respect to treatment process.

Structures, rooms and areas accommodating chemical storage and feeding equipment should be arranged to provide convenient access for chemical deliveries, equipment servicing and repair, and observation of operation. It is recommended that wherever possible the storage area be separated from the main plant and that segregated storage be provided for each chemical. Where two or more chemicals could react with undesirable effects, the drainage piping (if provided) from the separate chemical handling areas should not be interconnected. For dangerous materials such as gaseous chlorine, either floor drains in the storage and scale rooms should be omitted entirely, or floor drains installed, but kept totally separated from the drainage systems for the rest of the building.

Chemical storage areas need to be enclosed in dykes or curbs that are capable of containing 110 percent of the maximum stored volume until it can be safely transferred to alternate storage or released to the sewage at a controlled rate that will not damage facilities, inhibit treatment or contribute to receiver pollution. Curbs, sloped areas and drains should be used to contain spills at unloading areas.

It is strongly recommended that all chemical storage be at or above the surrounding grade. Where subsurface locations for chemical storage tanks are proposed, these locations should be free from sources of possible contamination, and positive drainage (i.e., pumped) for ground or storm waters, chemical spills and overflows needs to be assured. Where above grade storage is provided, due consideration should be given to the method of unloading chemicals; for example, there is a limit on the allowable pressures to be used for air-padded trucks. Where dry chemicals are used, it is recommended that some form of loading dock or ramp be provided.

The storage areas and locations should be arranged to prevent any chemical spills and to facilitate clean-up operations. The floor surfaces should be smooth, impervious, slip-proof and adequately sloped to drainage points.

Adequate measures should be taken to provide a minimum temperature of 15 °C (59 °F) for chlorine gas areas and the remainder of the chemical buildings should be heated to a temperature to prevent crystallization or freezing of chemical or abnormally high viscosities (polyelectrolytes) which would make pumping difficult.

The ventilation system for chemical buildings should be such that exhausted air is passed outside the building and arranged within the building to provide for slight negative pressures in areas where dry chemicals are in use, as a dust control measure. Where large amounts of dust are anticipated from chemical handling operations, adequate air filtration equipment should be included in the ventilation system.

The vents of tanks containing chemicals that are sensitive to moisture should be equipped with desiccant cartridges.

The designer should note that special precautions may be necessary in the design of air emissions control systems to prevent chemical concentrations at the *point of impingement* from exceeding limits permitted within the building or site by *Air Pollution-Local Air Quality*, (O. Reg. 419/05) made under the *Environmental Protection Act* (EPA) or which might be hazardous. An approval under Section 9 of the EPA is required if a contaminant may be discharged into the air from any part of the plant. For details, see the *ministry's Guide for Applying for Approval (Air and Noise)*.

All chemical buildings should be provided with eye wash units and/or deluge showers, adequate facilities for cleaning up chemical spills, space for cleaning and storage of the recommended protective equipment and adequate warning signs, conspicuously displayed where identifiable hazards exist. It is recommended that all doors in chemical rooms open outward and that corridors or space between storage areas be a minimum 1.5 m (5 ft) wide to permit the use of equipment such as hand trucks.

The use, storage and handling of any hazardous materials should be in accordance with federal and provincial legislation (e.g. Regulation 860, *Workplace Hazardous Materials Information System* (WHMIS), made under the *Occupational Health and Safety Act* as well as the *Building Code*, O. Reg. 350/06 made under the *Building Code Act, 1992* and the *Fire Code*, O. Reg. 388/97 made under the *Fire Protection and Prevention Act, 1997*).

Chemical buildings or storage areas should be provided with adequate warning signs, conspicuously displayed where identifiable hazards exist, a storage area for *Material Safety Data Sheets (MSDS)* and other provisions in accordance with the *Occupational Health and Safety Act* (OHSA), R.S.O. 1990, c O.1. All storage containers should be conspicuously labelled with a WHMIS label that includes: the product name, the supplier name, hazard symbol(s), risk, precautionary measures and first aid measures. An MSDS should be available for each chemical.

20.2.1 Liquid Chemicals

All storage tanks should be constructed of a material proven for the intended service. Since some chemicals, such as ferric chloride, are delivered at very high temperatures (up to 60 °C or 140 °F), the tanks and associated equipment should be able to withstand such temperatures. Tanks located outside should be heat traced and insulated to maintain the minimum allowable temperature for the chemical being stored.

All storage tanks should be provided with an adequate size fill line, minimum 50 mm (2 in) in diameter, which is sloped to drain into the tank. The fill line should be adequately identified at the end remote from the tank and provision should be made to drain this fill line, if a "down leg" exists.

Each tank should have an adequate vent line, minimum size 50 mm (2 in), with a down-turned end. Where venting outside the room is required, the vent should be provided with an insect screen.

All tanks should have an overflow adequate for the rate of fill proposed for the tank and overflows should be sloped down from the tanks, with ends turned down and having insect screens, and should have a readily visible free discharge directed to a suitable containment area. Tanks to be filled by pumping should be provided with an overflow not less than 300 mm (12 in) above the design level and not less than 150 mm (6 in) above the design level when filling is done by gravity flow.

Each tank should be provided with means to indicate the level of the contents in the tank and where an external level gauge is provided, it should have a shut-off valve at the tank connection. Each tank should be equipped with a drain. Tanks should have removable lids or covers or manholes where the contents are such that venting indoors is permitted. In the case of tanks which are to be vented outside, the covers or manholes should be constructed so as to be airtight. Overflows from tanks holding corrosive chemicals should be provided with seals to prevent vapours migrating to the room.

Tanks should be arranged to provide a minimum clear space all around them of not less than 300 mm (12 in). Where tanks with liners are used, weep holes should be provided in the outer shell to show positive indication of liner leakage.

A containment system should surround liquid storage tanks to contain spills, having a capacity exceeding the volume of all storage vessels contained (i.e., 110percent of overall content volume).

All storage tanks should be conspicuously signed with the contents and principal hazards of the contents shown.

20.2.2 Dry Chemicals

Where dry chemicals are to be used, provision should be made to minimize dust problems in handling. The use of granular materials is preferred.

Particular care should be taken with fine dusts around electrical equipment. Where exhaust fans, filters and vacuum conveying systems are used, grounding should be provided to prevent any static electricity build up.

20.2.3 Liquid-Gas Chlorine

Gas chlorination equipment (chlorinators, weight scales, chlorine cylinders) needs to be located in an isolated room or rooms. In larger installations the storage and weighing facilities should be in a room separate from the chlorinators. The construction of the facility should be of fire resistant and corrosion-proof material, have concrete floors and be gastight. All interior surfaces should be coated with a substance impermeable to chlorine gas.

Safety chains need to be used to retain each cylinder, in storage and on weigh scales, in a safe upright position. Chlorine should not be stored below ground level and the cylinders need to be protected from excessive heat, dampness and mechanical damage. One-tonne cylinders should be stored on their sides on level racks.

Areas containing chlorine or chlorination equipment need to be clearly marked. The exit doors with panic hardware needs to be hinged to open outwardly. There needs to be two or more exits if the distance to travel to the nearest exit exceeds 5 m (15 ft). All exits from the chlorine room and storage area should be to an outside wall. Access between these rooms is permitted if they have a common wall.

The temperature in the chlorine storage and scale room should not be higher and preferably slightly lower than that in the chlorinator room. The gas lines between the scales, chlorinators and injectors should not be located directly on an outside wall or in a location where low temperatures may be encountered.

As previously indicated, floor drains from chlorine storage or scale rooms should not be connected with drainage systems of other parts of the building or other buildings. Chlorine gas is heavier than air and could travel via drains, such as floor drains and foundation drains, into other rooms. If floor drains are to be used, they should be completely separated from other drainage systems. As an alternative to floor drains, the floors may be sloped towards doors to provide the needed drainage.

Gas detectors and alarms should be provided for storage and scale rooms. If the plant is not continuously manned or connected to a plant Supervisory Control And Data Acquisition (SCADA) system, the alarm should terminate at a fire station, police station or other 24-hour manned location.

Each sewage treatment works using liquid-gas chlorine should have a contingency plan to deal with major gas leaks.

Chlorine gas feed and storage rooms should be provided with inspection windows to permit viewing of the interior of the room and equipment. Switches for fans and light should be outside the room at the entrance and a signal light indicating fan operation should be provided at each entrance. Vents from feeders and storage should discharge to the outside atmosphere above grade and should slope down wherever possible.

20.3 CHEMICAL APPLICATION POINTS

All chemicals should be applied to the sewage streams or sludges at such points and in such a way as to ensure the maximum efficiency for treatment and to provide maximum safety to the operators. Chemicals should be added at a point of turbulence or at a point of mechanical or induced mixing or through a diffuser to ensure satisfactory mixing. Particular care should be taken where the point of addition is close to a point where the flows split. Alternate chemical addition points should be provided to give maximum flexibility of operation where appropriate. Where chemicals are added to lines under pressure, a suitable isolating valve should be provided.

With phosphorus removal chemicals, it is desirable to terminate the chemical feed point above the liquid level of the tank or channel so that the chemical discharge can be observed by the operator.

20.4 CHEMICAL FEED EQUIPMENT

Where the chemical added is necessary for the protection of the receiving waters such as chlorination, dechlorination, phosphorus removal or other critical processes, a minimum of two feeders should be provided, with one acting as a standby unit.

The design and *capacity* of feeders should be such that they will be able to supply at all times, the necessary amounts of chemicals at an accurate rate, throughout the feed range. Feeders should be capable of proportioning the chemical feed to the rate of sewage or sludge flow.

Chemical solutions can be prevented from being siphoned into the sewage stream by either assuring discharge at a point of positive pressure or providing vacuum relief or a suitable air gap.

All positive displacement pumps should be equipped with adequately sized pressure relief valves. If the pumped fluid is relieved through this valve, it needs to pass to a safe location, preferably back to the storage tank. Where liquid-filled diaphragm pumps are in use, the over-pressure should be relieved internally or by discharge of the motive fluid to a safe location. Pressure relief valves should be set at a safe relief pressure to avoid damage to the chemical feed lines.

Chemical feed lines should be kept as short as possible, protected against freezing and readily accessible through their entire length.

The minimum line size should be 12 mm (0.5 in), unless the material pumped exhibits scale forming tendencies, then the minimum size should be 25 mm (1 in). In general, the feed line should be designed to be consistent with the scale forming or solids depositing properties of the material conveyed. Where feed lines are provided from duplicate pumping units or passed to a distribution manifold, adequate valving should be provided to isolate sections of the line.

Where reciprocating type pumps are to be used, it is recommended that flexible connections be provided on the pump suction and discharge to prevent the transmission of vibrations to the feed line. These flexible connections should be sufficiently rigid to withstand both the pump suction and discharge pressure, and reinforced hose is recommended. The pump, in combination with its suction piping and valving arrangement, should be such that the pump discharge rate remains the same regardless of the level of chemical in the feed tank.

Where dry chemical feeders and storage equipment are provided, the design of the storage equipment should be adequate to prevent bridging or other problems in the storage silo. Dry chemical feeders will be acceptable if they measure either volumetrically or gravimetrically and provide for effective solution of chemical in a solution pot. The dry chemical feed system should be enclosed to prevent emission of chemical dusts into the operating room.

Feeders may either be manually or automatically controlled and automatic control should revert to manual control as necessary. Feed rates proportional to flows should be provided.

Where solution tanks are to be used, the designer should provide as means to maintain a uniform strength solution. Continuous agitation should be provided to maintain slurries in suspension. Normally, two solution tanks will be required to ensure continuity of supply in servicing the solution tank. Each tank should be provided with a drain.

Make-up water for the solution tank should enter the tank not less than 150 mm (6 in) above or two pipe diameters above the maximum solution level, whichever is greater.

Where the design of the chemical feed system includes day tanks, such day tanks should have a maximum capacity equivalent to the chemical consumed over a 30-hour period. Day tanks should either be scale-mounted or have a calibrated level gauge provided. The piping arrangement for refilling the day tanks should be such that it will prevent over-filling of the tank. In all other respects, the requirements for day tanks should conform to the requirements for bulk storage tanks.

20.5 OPERATOR SAFETY

The design of *sewage works* need to include provisions to protect operator and other worker safety and health. The safety of workers and workplaces is governed by the *Occupational Health and Safety Act* and the *Workplace Safety and Insurance Act*, as well as the regulations made under these acts, which are both administered by the Ontario Ministry of Labour.

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CHAPTER 21

CONTROL AND TREATMENT OF COMBINED SEWER OVERFLOWS

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CONTROL AND TREATMENT OF COMBINED SEWER OVERFLOWS

This chapter provides guidance regarding combined sewer overflow (CSO) control as well as an overview of minimum treatment requirements. A discussion of source management and commonly used treatment technologies is also provided. Many of the control options presented in this chapter are operational measures which may serve to mitigate the impacts associated with combined sewer systems.

21.1 CSO CONSIDERATIONS

21.1.1 CSO Control Requirements

The designer should consider the requirements of the *ministry* Procedure F-5-5, *Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems* in the design or upgrading of *sewage works* associated with combined sewer systems. The minimum CSO controls in accordance with this Procedure should consist of the following:

- Elimination of CSOs during dry-weather periods except under emergency conditions;
- Establishment and implementation of Pollution Prevention programs that focus on pollutant reduction activities at source, as described in the Procedure;
- Establishment and implementation of proper operation, regular inspection and maintenance programs for combined sewer systems;
- Establishment and implementation of a floatables control program to control coarse solids and floatable materials;
- Maximizing the use of the collection system for the storage of *wet weather flows* which are conveyed to the sewage treatment plant (STP) for treatment when *capacity* is available (e.g. by adjusting regulator settings);
- Maximizing flow to the STP for the treatment of wet weather flows (e.g. by removing obstructions to flow), with the secondary treatment capacity at the STP being utilized to the maximum extent possible for treating these flows; and
- Capture and treatment of all *dry weather flow* plus 90 percent of the volume resulting from wet weather flow that is above the dry weather flow for an average year, during a seven-month period commencing within 15 days of April 1, as specified in the Guideline.

21.1.1.1 Beach Protection

Additional controls above the minimum CSO controls described above are required for swimming and bathing beaches affected by CSOs. The designer should refer to Procedure F-5-5 for more information.

21.1.1.2 New Sanitary Connections to Combined Sewer Systems

New developments tributary to combined sewer systems should be avoided until the necessary upgrades to the combined sewer system can be completed. Conditions under which some development will be permitted are specified in Procedure F-5-5.

21.1.1.3 New Storm Connections to Combined Sewer Systems

As stated in Procedure F-5-5, "new storm drainage system should not connect to existing combined sewer systems if that increases the gross area serviced by the combined sewer system except where evaluations indicate that circumstances allow no other practical alternative. The evaluations should be documented as part of a Pollution Prevention and Control Plan."

21.1.2 Treatment Levels

The treatment of wet weather flows from combined sewer systems may occur at the central STP or at other locations such as satellite treatment facilities. Minimum treatment requirements are described in Procedure F-5-5.

21.1.3 CSO Monitoring

The designer, in consultation with the municipality or operating authority of the combined sewer system, should consider the provision for monitoring equipment for sewage flows and overflows at various locations within the sewer system for the purposes of assessing upgrading requirements and determining compliance with ministry requirements.

21.1.4 CSO Modelling

The following subsections briefly summarize combined sewer system modelling objectives, model selection strategies and model development and application, including model calibration and validation and the different types of model simulations (e.g. long-term continuous versus storm event simulations). A more detailed discussion is provided in the ministry *Stormwater Management Planning and Design Manual* (2003) as well as the U.S. EPA guidance documents *Combined Sewer Overflows Guidance for Long-Term Control Plan* and *Combined Sewer Overflows Guidance for Monitoring and Modeling*.

21.1.4.1 Combined Sewer System Modelling Objectives

The primary objective of combined sewer system modelling is to understand the hydraulic response of the combined sewer system to a variety of precipitation and drainage area inputs. Combined sewer system modelling can also be used to predict pollutant loadings to receiving waters. Once the model is calibrated and verified, it can be used for numerous applications that support CSO planning efforts, including:

- Predicting overflow occurrence, volume and in some cases, quality for rain events other than those which occurred during the monitoring phase. These can include a storm event of large magnitude (long recurrence period) or more frequent storm events over an extended period of time;
- Predicting the performance of portions of the combined sewer system that have not been extensively monitored;
- Developing CSO statistics, such as annual number of overflows and percent of combined sewage captured;
- Optimizing combined sewer system performance. In particular, modelling can assist in locating storage opportunities and hydraulic bottlenecks and demonstrate that system storage and flow to the STP are maximized; and
- Evaluating and optimizing control alternatives. The model can be used to evaluate the resulting reductions in CSO volume and frequency for both simple and complex control strategies.

For a specific system, the designer should consider whether analysis using complex computer models is needed. In simple systems, computation of hydraulic profiles using basic equations and spreadsheet programming might be sufficient for identifying areas where certain measures can be implemented and for evaluating their hydraulic effect. In complex combined sewer systems, that have looped networks or sections that surcharge, a hydraulic computer model is a useful tool.

Continuous simulation refers to the use of long-term rainfall records (e.g. from several months to several years) rather than rainfall records for individual storms (i.e., design storms). Continuous simulation has several advantages:

- Simulations are based on a sequence of storms so that the additive effect of storms occurring close together can be examined;
- Storms with a range of characteristics are included; and
- Long-term simulations enable the development of performance criteria based on long-term averages, which are not readily determined from design storm simulations.

21.1.4.2 Combined Sewer System Model Selection

Combined sewer system modelling involves two distinct elements – hydraulics and water quality:

- Hydraulic modelling consists of predicting flow characteristics in the combined sewer system. These characteristics include the different flow rate components (i.e., sanitary sewage, infiltration and stormwater runoff), the flow velocity and depth in the interceptors and sewers and the CSO flow rate and duration; and

- Combined sewer system quality modelling consists of predicting the quality of the combined sewage in the system, particularly at CSO outfalls and at the treatment plant. Water quality is measured in terms of critical parameters, such as bacterial counts and concentrations of BOD_5, suspended solids, nutrients and toxic contaminants.

Some models include both hydraulic and water quality components while others are limited to one or the other. The designer, in consultation with the system owner/operator, should determine which type of model is most appropriate.

21.1.4.3 Model Calibration and Verification

A general model should be adjusted to the characteristics of a specific site and situation. Modelers use model calibration and verification first to perform this adjustment and then to demonstrate the credibility of the model simulation results. Using an uncalibrated model (e.g. using typical industry values for model parameters) might be acceptable for screening purposes. Without supporting evidence, however, the uncalibrated result might not be accurate. To use model simulation results for evaluating control alternatives, the modeller should provide evidence demonstrating the model's reliability.

21.2 STORMWATER MANAGEMENT

The ministry *Stormwater Management Planning and Design Manual* provides technical and procedural guidance for the planning, design and review of stormwater management practices. Specifically, the manual provides guidance regarding:

- Environmental planning;
- Environmental design criteria;
- Stormwater management plans and stormwater management practice (SWMP) design;
- Infill development; and
- Operation, maintenance and monitoring.

The designer should consult the ministry manual for guidance regarding the development of a stormwater management program and for the design of SWMPs, as part of an overall CSO control program.

21.3 SOURCE MANAGEMENT AND CONTROL TECHNOLOGIES

Control measures can include technologies, operating strategies, public policies and regulations, or other measures that would contribute to some aspect of CSO control. Control measures can generally be classified under one of the following categories:

- Source controls;
- Collection system controls;
- Storage technologies; and
- Treatment technologies.

A brief description of some of the options available for each category is provided below.

21.3.1 Source Control

Source controls affect the quantity and/or quality of runoff that enters the collection system. Since source controls reduce the volumes, peak flows or pollutant loads entering the collection system, the size of more capital-intensive downstream control measures can be reduced or, in some cases, the need for downstream facilities can be eliminated. The source controls discussed below include both quantity and quality control measures. A more detailed discussion is provided in the U.S. EPA document *Combined Sewer Overflows Guidance for Long-Term Control Plan*.

21.3.1.1 Porous Pavements

Porous pavements reduce runoff by allowing stormwater to drain through the pavement to the underlying soil. Porous pavements, most commonly used in parking lots, require skill and care in installation and maintenance to ensure that the pores in the pavement do not become plugged.

21.3.1.2 Flow Detention

Detention ponds in upland areas and roof-top storage can store stormwater runoff temporarily, delaying its introduction into the collection system and thereby helping to attenuate peak wet weather flows in the collection system. The detention facilities drain back to the collection system when peak wet weather flows subside.

21.3.1.3 Area Drain and Roof Leader Disconnection

In highly developed areas with relatively little open, pervious space, roof leaders and area drains are commonly connected directly to the combined sewage collection system. Rerouting of these connections to separate storm drains or available pervious areas can help reduce peak wet weather flows and volumes.

21.3.1.4 Foundation Drains

Foundation drains may be connected to storm sewers in some areas. Sump pumps may be used to discharge foundation drainage to the surface or to soakaway pits. In areas where the seasonal high water table is within 1 m (3 ft) of the foundation drain, sump pumps should not be used in order to prevent the formation of a looped pumping operation and excessive pump operation. Where sump pumps are not feasible, a “third pipe” may be used to convey foundation drainage to a suitable discharge point.

21.3.1.5 Use of Pervious Areas for Infiltration

Detention of storm flow in pervious areas not only helps attenuate peak wet weather flow in the collection system but also reduces runoff volume through infiltration into the soil. Grassed swales, infiltration basins and subsurface leaching facilities can be used to promote infiltration of runoff. Infiltration sumps can be used in areas with well draining soils. The designer should consider the impact of cold climate conditions. This type of control might be more appropriate as a requirement for future development or re-development and could be implemented through sewer use by-laws and through strict review of proposed development plans.

21.3.1.6 Solid Waste Management

Although littering is generally prohibited, it is a common problem in many communities. Street litter typically includes metallic, glass and paper containers, cigarettes, newspapers and food wrappers. If not removed from the street surfaces by cleaning equipment, some of these items often end up in combined sewer overflows (*Section 8.5.6.4 - Bypasses and Overflows at Sewage Works Serving Combined Sewers*), creating visible pollution due to their floatable nature.

Enforcement of anti-litter by-laws, public education programs, and conveniently placed waste disposal containers might be effective, low-cost alternatives, especially in urban business areas. The proper disposal of leaves, grass clippings, crankcase oil, paints, chemicals and other such wastes can be addressed in a public education program. Because the results of such a program depend on voluntary cooperation, the level of effectiveness can be difficult to predict.

21.3.1.7 Street Sweeping

Frequent street sweeping can prevent the accumulation of dirt, debris and associated pollutants, which may wash off streets and other tributary areas to a combined collection system during a storm event. Current sweeping practices can be analyzed to determine whether more frequent cleaning will yield CSO control benefits. The overall effectiveness of street sweeping as a CSO control measure depends on a number of factors, including frequency of sweeping, size of particles captured by sweeping, street parking regulations and climatic conditions, such as rainfall frequency and season.

21.3.1.8 Fertilizer and Pesticide Control

Fertilizers and pesticides washed off the ground during storms contribute to the pollutant loads in stormwater runoff. It is important that all users follow proper handling and application procedures. The use of less toxic formulations should also be encouraged. Because most of the problems associated with these chemicals are a result of improper or excessive usage, a public education program may be beneficial.

21.3.1.9 Snow Removal and De-Icing Control

This abatement measure involves limiting the use of chemicals for snow and ice control to the minimum necessary for public safety. This, in turn, would limit the amount of chemicals (i.e., normally salt) and sand washed into the collection system and ultimately contained in CSOs. Proper storage and handling measures for these materials might also reduce the impacts of runoff from material storage sites.

21.3.1.10 Soil Erosion Control

Controlling soil erosion is important in relation to CSOs and water quality for a number of reasons:

- Soil particles carry nutrients, metals, and other toxics which may be released into the receiving water, contributing to algal blooms, potential toxic effects and bioaccumulation of toxics; and
- Eroded soil can contribute to sedimentation problems in the collection system, potentially reducing hydraulic capacity.

Properly vegetated and/or stabilized soils are not as susceptible to erosion and thus will not be washed off into combined sewers during wet weather. Like for fertilizer and pesticide control, an educational program may be useful in controlling soil erosion. Implementation and enforcement of erosion control regulations at construction sites can also be effective.

21.3.1.11 Commercial/Industrial Runoff Control

Commercial and industrial lands, including gasoline stations, railroad yards, freight loadings areas and parking lots contribute grit, oils, grease and other pollutants to combined sewer systems. Such contaminants can run off into receiving waters. Installing and maintaining oil/grit separators in catchbasins and area drains can help control runoff from these areas, while pretreatment requirements can be identified as part of the municipality's sewer use by-laws.

21.3.1.12 Animal Waste Removal

This measure refers to removing animal excrement from areas tributary to combined sewer systems. The impact of this control measure is difficult to quantify; however, it might be possible to achieve a minor reduction in bacterial load and oxygen demand. This best management practice (BMP) can be addressed by a public information program and "pooper-scooper" by-laws.

21.3.1.13 Catchbasin Cleaning

The regular cleaning of catchbasins can remove accumulated sediment and debris that could ultimately be discharged in CSOs. In many communities, catchbasin cleaning is targeted more toward maintaining proper drainage system performance than pollution control.

21.3.2 Combined Sewer System Control

Combined sewer system controls and modifications affect CSO flows and loads for stormwater runoff that has entered the collection system. This category of control measures can reduce CSO volume and frequency by maximizing the volume of flow stored in the collection system, or maximizing the capacity of the system to convey flow to a STP and includes the control alternatives described in the following subsections. A more detailed discussion is provided in the U.S. EPA guidance document *Combined Sewer Overflows Guidance for Long-Term Control Plan*.

21.3.2.1 Sewer Line Flushing

Sediments that accumulate in sewers during dry weather can be a source of CSO contaminants during storm events. Periodically flushing sewers during dry weather conditions will convey settled materials to the STP. The cost effectiveness of such a program, however, depends on treatment, labour costs, physical sewer characteristics and productivity.

Sewer cleaning usually requires the use of a hydraulic, mechanical or manual device to re-suspend solids into the sewage flow and carry them out of the collection system. This practice might be more effective for sewers with very flat slopes. Cleaning costs increase substantially for larger interceptors due to occasional accumulations of thick sludge blankets in inverts.

Grit management should be considered and precautions taken to avoid overloading the STP during the cleaning process. Consideration should also be given to disposal of debris dislodged during flushing. Additional information is provided in the ministry *Stormwater Management Planning and Design Manual* (2003).

21.3.2.2 Maximizing Use of Existing System

This control measure involves maximizing the quantity of flow collected and treated, thereby minimizing CSOs It involves ongoing maintenance and inspection of the collection system, particularly flow regulators and tidegates. In addition, minor modifications or repairs can sometimes result in significant increases in the volume of storm flow retained in the system. Strict adherence to a well-planned preventive maintenance program can be a key factor in controlling dry and wet weather overflows.

21.3.2.3 Sewer Separation

Separation is the conversion of a combined sewer system into separate stormwater and sanitary sewage collection systems. Sewer separation is a positive means of eliminating CSOs and preventing sanitary flow from entering the receiving water during wet weather periods and may be applicable and cost-effective on a site-specific basis. The benefits of separation should be evaluated, with consideration given to a potential increase in the loading of stormwater runoff pollutants (e.g. sediment, bacteria, metals, oils) being discharged to the receiving water, cost (it is relatively expensive) and the potential disruption of traffic and other community activities during construction.

21.3.2.4 Infiltration/Inflow Control

Excessive infiltration and inflow (I/I) can increase operation and maintenance costs and can consume hydraulic capacity, both in the collection system and at the STP. In combined sewer systems, surface drainage is by design the primary source of inflow. Sources of inflow in combined sewer systems should be controlled, including roof leaders, sump pumps, and tidal inflow (i.e., through leaking or missing tidegates, where applicable).

Infiltration is *groundwater* that enters the collection system through defective pipe joints, cracked or broken pipes, manholes, footing drains and other similar sources. Infiltration flow tends to be more constant than inflow. Significant lengths of sewers usually need to be rehabilitated to effectively reduce infiltration and the rehabilitation effort should include house laterals.

Implementation of an effective maintenance and inspection program, consisting of closed-circuit television (CCTV) inspections, manhole and lateral assessments, is necessary to control infiltration.

21.3.2.5 Regulating Devices and Backwater Gates

Flow regulating devices have been used for many years in combined sewer systems to direct dry weather flow to interceptors and to divert wet weather combined flows in excess of interceptor capacity to receiving waters (i.e., as CSOs).

In general, regulators fall into two categories: static and mechanical. Static regulators have no moving parts and, once set, are usually not readily adjustable. They include side weirs, transverse weirs, restricted outlets, swirl concentrators (i.e., flow regulators/solids concentrators) and vortex valves. Mechanical regulators are adjustable and might respond to variations in local flow conditions or be controlled through a remote telemetry system. They include inflatable dams, tilting plate regulators, reverse-tainter gates, float-controlled gates and motor-operated or hydraulic gates.

The designer should also take into account maintenance issues (i.e., frequency and complexity) associated with the particular type of regulator being considered.

21.3.2.6 Real-Time Control

System-wide real-time control (RTC) programs can provide integrated control of regulators, outfall gates and pump station operations based on anticipated flows from individual rainfall events, with feed-back control adjustments based on actual flow conditions within the system. Computer models associated with the RTC system allow an evaluation of expected system response to control commands before execution. Localized RTC might also be provided to individual dynamic regulators, based on feedback control from upstream and/or downstream flow monitoring elements. As with any plan for improving in-line storage, to take the greatest advantage of RTC, a combined sewer system should have relatively flat upstream slopes and sufficient upstream storage and downstream interceptor capacity.

21.3.2.7 Flow Diversion

Flow diversion is the diversion or relocation of dry weather flow, wet weather flow, or both from one drainage basin to another through new or existing drainage basin interconnections. Flow diversion can relieve an overloaded regulator or interceptor reach, resulting in a more optimized operation of the collection system. Flow diversion can also be used to relocate combined sewer flow from an outfall located in a more sensitive receiving water area to an outfall located in a less sensitive one.

21.3.3 Storage

Wet weather flows can be stored for subsequent treatment at the STP once treatment and conveyance capacity have been restored. Specific design guidance for storage facilities is provided in the Environment Canada *CSO Treatment Technologies Manual*.

21.3.3.1 In-Line Storage

In-line storage is storage in series with the sewer. In-line storage can be developed in two ways:

- Construction of new tanks or oversized conduits to provide storage capacity; or
- Construction of a flow regulator to optimize storage capacity in existing conduits.

The new tanks or oversized conduits are designed to allow dry weather flow to pass through, while flows above a design peak is restricted, causing the tank or oversized conduit to fill. A flow regulator on an existing conduit functions under the same principle, with the existing conduit providing the storage volume. Developing in-line storage in existing conduits is typically less costly than other, more capital-intensive technologies, such as off-line storage/sedimentation and is attractive because it provides the most effective utilization of existing facilities. The applicability of in-line storage, particularly the use of existing conduits for storage, is very site-specific, depending on existing conduit sizes and the risk of flooding due to an elevated hydraulic grade line.

21.3.3.2 Off-Line Near Surface Storage

This technology reduces overflow quantity and frequency by storing all or a portion of diverted wet weather combined flows in off-line storage tanks. The storage arrangement is considered to be parallel with the sewer. Stored flows are returned to the interceptor for conveyance to the STP once system capacity is available. In some cases, flows are conveyed to a CSO treatment facility.

21.3.3.3 Deep Tunnel Storage

This technology provides storage and conveyance of storm flows in large tunnels constructed well below the ground surface. Tunnels can provide large storage volumes with relatively minimal disturbance to the ground surface, which can be very beneficial in congested urban areas. Flows are introduced into the tunnels through dropshafts and pumping facilities are usually required at the downstream ends for dewatering.

21.3.4 Treatment Technology Options

The design of CSO treatment facilities involves all the normal considerations associated with any STP design plus the rather unique aspect of an intermittent and highly variable influent. In addition, special attention should be paid to siting issues and the potential operating impacts such as odours that may be associated with a satellite facility.

Most CSO treatment facilities are comprised of a number of unit processes tied together in a train. Multiple processes are employed because of the specialized function of each unit (e.g. solids removal, disinfection). Some processes such as screening and degritting are employed primarily to protect downstream process equipment. Even so, screens and degritting serve also to remove gross solids and heavy grit particles, which enhance the quality of the treated effluent. Some unit processes, such as UV disinfection, require more extensive pretreatment to be viable. UV disinfection design guidelines are provided *in Section 14.4 - Ultraviolet Irradiation*. A complete CSO treatment process train should address both liquid and solids treatment and residue disposal.

The location of the proposed CSO treatment facility should also be considered in finalizing the process train. Satellite facilities should incorporate all liquid train unit processes on site. Depending upon circumstances it may not be desirable to utilize a process requiring chemical storage at some satellite locations. Community sensitivity may in this case dictate a different process train selection.

Other design considerations may also influence process train selection. For example, the facility footprint may be a major factor where limited siting opportunities are available. In turn, this may favour very high rate treatment, integrating a number of unit processes within a single facility. Other processes are complex to operate and require skilled attention. These may not be well suited to satellite locations. Finally, both capital and operating costs play a major role in technology selection and should be carefully weighed during process train evaluation.

Specific design parameters, such as flow rate, hydraulic loading and solids loading for a chosen technology will depend on the treatment objectives and the characteristics of the specific unit selected. These parameters are often established through piloting and in consultation with the manufacturer.

Additional information is also provided in Environment Canada's *CSO Treatment Technologies Manual*.

21.3.4.1 Screening Devices

Screening technologies can be applied in a CSO treatment train in one of two modes:

- Part of an overall CSO treatment process train. This application is associated with CSO treatment facilities capable of providing "primary equivalent treatment". The screening device can be employed either for pretreatment or for effluent polishing; and
- Stand-alone where the screen is the main treatment device although not generally able to provide "primary equivalent treatment". This application is used to address gross solids including floatable materials in the remaining overflows discharging without "primary equivalent treatment" or CSO treatment facility bypass streams.

There are a wide variety of screens and screening devices in all manner of configurations, aperture sizes and applications. Therefore, the designer should consult the manufacturer for specific design guidance.

21.3.4.2 Floatables Control

Several technologies are available for floatables control, including screens, fabric nets, rotary sieves, or by using systems operating on the vortex principle, traps in catchbasins and simple underflow baffles in the overflow path.

21.3.4.3 Ballasted Clarifiers

Ballasted clarification is a term applied to proprietary technologies that employ ballasted coagulation-assisted settling. The main advantage of these processes is the very high rate of treatment possible that allows a reasonably small footprint. Because of the typically high coagulant and coagulant aid dosages employed, these technologies may also yield a greater degree of pollutant removal.

These technologies may be employed at satellite locations or as part of a stand alone or integrated facility at a STP. The designer should consult with the manufacturer regarding specific design requirements. Pilot testing is recommended for site-specific applications. For additional information on ballasted clarifiers see *Section 15.5 – High Rate Clarification*.

21.3.4.4 Retention Treatment Basins (Assisted and Unassisted)

Retention treatment basins (RTB) are intended to provide removal of CSO pollutants through sedimentation, removal of floatable materials (i.e., through integral baffling and screening) and full or partial capture of CSO discharges.

RTBs typically consist of several compartments to allow smaller overflow events to be captured and/or treated without the utilization of the entire facility. Dividing the storage volume of the RTB into separate compartments also allows different portions of the tank to be used for other unit operations, such as disinfection. Positively buoyant materials are usually removed by some type of baffle or skimmer arrangement or by installation of screens. Smaller storm events and portions of larger events are captured within the storage volume afforded by an RTB.

Specific guidance for the design of RTBs is provided in the Environment Canada *CSO Treatment Technologies Manual*.

21.3.4.5 Inclined Plate Settlers

Installation of inclined plate separators may increase the effective settling area and allowable surface overflow rate of a sedimentation unit. This technology should be combined with chemical treatment if a significant increase in suspended solids removal is needed. Plate settlers are generally incorporated into other process units, such as RTBs or ballasted clarifiers.

21.3.4.6 Flow Balance Methods

The flow balance technique is a retention treatment system using weighted plastic curtains hung from pontoons in shore locations in natural receiving water bodies. During operation, extraneous fluid (stormwater/CSO) flows into a pontooned multi-celled system while displacing currently stored fluid (lake water). This method relies on stratification of natural water and CSOs because of their different specific gravities. In freshwater bodies, multiple cells can be used in place of a bottom outlet.

After the storm event, the reverse pattern is created by pumping contents from the first or influent cell of the storage facility back to the sewer for subsequent treatment. Deposited solids can also be retrieved by pumping. For a small runoff event, only one or more of the cells are used. For a large event, all cells are filled and the system can be overflowed. Relative efficiency in transferring flows through plug-flow operation is determined by the number of cells and relative placement.

A feed pump and the discharge point are connected to the first compartment. Excess stormwater flow is diverted through large openings in intermediate baffles. Openings are placed alternatively at the bottom and top to prevent stratification caused by water temperature differences. The baffles, made of plastic cloth, hang from pontoons. The cloth is attached to the bottom with weights. There is no demand for absolute tightness against the bottom, as the sole function of the baffles is to create plug-flow conditions.

Additional information is provided in Water Environment Federation, *WEF Manual of Practice FD-17 Prevention and Control of Sewer System Overflows*. The designer should consult with the manufacturer regarding specific design requirements.

21.3.4.7 Vortex Separators (Assisted and Unassisted)

Vortex separators can be applied in satellite facilities or as part of an integrated or stand-alone facility at a STP. The separators are as versatile as RTBs and can be employed with the following configurations:

- In-vessel coagulant addition;
- In-vessel chemical disinfection;
- Integral fine screens; and
- Add-on disinfection (chemical or UV).

Separators do not necessarily require pretreatment although they are most often preceded by coarse screens. Fine screens (screens can be add-on or integral) can also be added if removal of neutral density floatables is required. Separators produce significant quantities of underflow which need to be transported and treated at a STP. Additional information on Vortex separators, as they are applied to preliminary treatment, can be found in *Section 10.3.3.4 - Vortex Grit Removal*.

The designer should consult with the manufacturer regarding specific design requirements. Pilot testing is recommended for site-specific applications.

21.3.4.8 Compressible Filter Media

A compressible filter media system is a proprietary technology that provides a high rate of solids removal through the use of synthetic fibre spheres. These processes are typically used as a polishing step in a more complex CSO treatment train, which may include other physical separation technologies upstream and UV disinfection downstream of the unit. Influent to the filters usually requires pretreatment to remove heavy solids and coarse floatable materials.

In theory, these units could be employed at satellite locations. However, until more experience is gained with automated operation, it is suggested that they be applied either as an integrated or stand-alone CSO treatment at STP locations. The designer should consult the manufacturer for specific design guidance. Pilot testing is recommended for site-specific applications.

21.4 HANDLING AND DISPOSAL OF CSO SOLIDS RESIDUALS

Residue management is a principal concern when using screening and degritting to treat CSOs. The handling of collected materials on trash racks and manually cleaned bar screens involves removing larger debris that may be collected, since the spacing will allow most of the floatables to pass through the openings. As the screen aperture decreases, there will be an increase in the amount of collected materials requiring removal. In addition, material collected immediately upstream and downstream of the screen may require periodic removal.

Typical volumes of residue removed by screens range from 3.5 to 84 L per 1,000 m^3 (0.47 to 11.2 ft^3/ (million US gal)) of influent. The actual amount of residue to be disposed of will vary depending on the following factors:

- Drainage system configuration;
- Time of year;
- Storm intensity and inter-event time;
- Velocity of flow through the screen; and
- Screen aperture.

The disposal of screenings may be provided by any of the following:

- Removal by hauling to disposal sites (landfills);
- Incineration, either alone or in combination with sludge and grit;
- Disposal with municipal solid wastes; and
- Return to sewage, either directly or via grinders and macerators.

To minimize handling and disposal costs, it is preferable to have screenings and grit returned to the sewer. Alternatively, screenings can be discharged to a container for ultimate disposal at both satellite and STP locations. Screening residues can also be returned to an interceptor sewer at satellite locations. In this case, the screenings will be once more removed at the STP and then taken to disposal.

Residuals management for screening and degritting operations is addressed in the Environment Canada *CSO Treatment Technologies Manual* as well as in *Section 10.6 - Screenings, Grit Handling and Disposal*.

21.5 DISINFECTION

Effluent disinfection is required where CSO affects swimming and bathing beaches and other areas where there are public health concerns. The interim effluent quality criterion for disinfected combined sewage during wet weather is a monthly geometric mean not exceeding 1000 *E. coli* organisms per 100 mL. This criterion may be modified by the Regional staff of the ministry on a case-by-case basis due to site-specific conditions.

All overflows and bypasses at the STP should be subjected to the disinfection process where available in order to reduce the bacterial loadings at discharge.

Specific guidance regarding the design of disinfection systems is provided in *Chapter 14 - Disinfection* as well as in the Environment Canada *CSO Treatment Technologies Manual*.

21.5.1 Chlorination and Dechlorination

In cases where chlorination is used as the disinfection process, subsequent dechlorination of the sewage works effluents should be used to minimize the adverse effects of chlorine residuals on public health and the aquatic environment where necessary.

21.5.2 Ultraviolet Irradiation

Ultraviolet (UV) irradiation for the disinfection of CSO should be used in conjunction with other unit processes capable of meeting primary-equivalent treatment requirements which should address floatables, TSS and $CBOD_5$. UV irradiation should follow pretreatment and solids separation processes. UV irradiation can be applied in satellite or STP integrated or stand-alone facilities.

CHAPTER 22

LARGE SUBSURFACE SEWAGE DISPOSAL SYSTEMS

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LARGE SUBSURFACE SEWAGE DISPOSAL SYSTEMS

This chapter provides an overview of large subsurface sewage disposal systems with design flows greater than 10,000 L/d. It also provides a comparison of these systems to those smaller systems regulated by Part 8 of Division B of the *Building Code* (O. Reg. 350/06) made under the *Building Code Act*, *1992*.

22.1 APPLICABLE LEGISLATION

Small (i.e., total daily design sanitary sewage flow of 10,000 L/d (2640 US gal/d) or less) individual or multiple subsurface sewage disposal systems, located wholly within the boundaries of the lot or parcel of land on which are located the residence(s), building(s) or facility/ies which they serve, are subject to the requirements of Part 8 of Division B of the *Building Code* (O. Reg. 350/06) made under the *Building Code Act, 1992*. This Act is administered by the Ontario Ministry of Municipal Affairs and Housing.

Under Part 8 of the *Building Code*, the means to determine the total daily design sewage flow are provided in Article 8.2.1.3. The values in Tables 8.2.1.3.A. and 8.2.1.3.B. represent sewage flow generation rates from residential occupancies and other specific facilities.

The design and construction of small subsurface sewage disposal systems, under the jurisdiction of the *Building Code Act*, *1992*, should strictly adhere to standards contained in Part 8 of the *Building Code* relating to:

- Classification of sewage systems and site evaluation;
- Sewage design flows and clearance requirements;
- Types and design of tanks used to collect, treat, hold sanitary sewage; and
- The sewage subsurface disposal design, construction, operation and maintenance requirements.

All *sewage works* with a design *capacity* in excess of 10,000 L/d, including subsurface disposal systems, are subject to the requirements of Section 53 of the *Ontario Water Resources Act* (OWRA) administered by the Ontario Ministry of the Environment[1]. Subsurface disposal systems with a design capacity in excess of 10,000L/d are referred to as large subsurface sewage disposal systems (LSSDS).

The design of a LSSDS under OWRA jurisdiction is subject to the *ministry* engineering review and approval process (*Section 1.5 – Ministry Approval*

[1] To clarify it further: (a) if a sewage system has a rated capacity of greater than 10,000 L/d (2,640 US gal/d), it is an OWRA sewage works regardless of location; (b) if a single property contains several small systems (each rated at less than 10,000 L/d (2,640 US gal/d)) but the combined rated capacity of the systems exceeds 10,000 L/d (2,640 US gal/d), all those systems are OWRA sewage works regardless of their individual capacity; and (c) if the system is not contained entirely within the property of the building (or buildings) it serves, it is an OWRA sewage works regardless of the capacity of the system.

Program for Sewage Works). The engineering review provides an evaluation of how the designer intends to meet the site-specific performance criteria established for the design life of the sewage works. The performance-based ministry engineering review is not prescriptive in nature and the required performance levels may be met using many alternatives, including new and innovative technologies.

The designer of the LSSDS is advised to consider, where appropriate and applicable, the design standards for small subsurface disposal systems contained in Part 8, Division B, of the *Building Code*. However, clearances or separation distances from large systems to such features as wells, *surface water* bodies and property boundaries need to be determined on a case-specific basis - see *Section 22.5 - Assessment of Impact on Water Resources*.

Ministry technical reviewers will examine the proponent's assessment of effects on water resources. This review evaluates the hydrogeological aspects of the assessment such as the subsurface conditions at the site, the choice of "reasonable use" *groundwater* criteria at the downgradient property boundary, and the site-specific discharge criteria to ensure that the downgradient criteria are met. In cases where discharge of groundwater to surface water is a concern, a surface water assessment and review may also be required.

Depending on the location of the site, the *Clean Water Act*, associated regulations and source water planning may apply. If the proposed system is to be sited in a location within a source water protection "vulnerable area" as defined under the *Clean Water Act*, the designer is advised to consider and address the requirements of the *Clean Water Act* prior to proceeding with an OWRA application for approval. Consultation with the local Conservation Authority or Source Water Protection Authority is recommended to determine if this is a concern and if so what specific requirements need to be addressed.

22.2 GENERAL

In addition to the common use of subsurface sewage disposal (i.e., septic tank systems) for individual residences, there are many applications that use this method of sewage disposal for large buildings or a number of buildings. Large sewage systems of this type may include fairly extensive sewage collection systems and sewage treatment works and may be similar to systems from which the final effluent is discharged to surface water.

Some typical examples of large systems are those serving:

- Nursing homes, hotels, motels and institutions;
- Subdivisions;
- Mobile home parks and tent and trailer parks;
- Clubhouses;
- Churches;
- Recreational parks and centres;

- Industrial and commercial parks, establishments and plazas; and
- Residential condominiums where each sewage system serves several units.

Proposals of LSSDS are often considered due to non-availability of a municipal sewer system. Where the municipal sewer services are in an adjacent area, but require either sewer extension or treatment plant expansion (or both) in order to service the new development, the designer should ensure that the proposal is not in conflict with zoning by-laws or the Official Plans for the area.

While this guideline is not intended as a statement on land use, it is acknowledged that most applications of LSSDS are in areas not likely to be serviced by centralized municipal or communal sewage works in the future. When considering the use of large subsurface systems in these areas, it is especially important to take into account the individual and cumulative effects of existing and future uses of the adjacent land on the operation of the systems.

The sanitary servicing strategy for a large development should be based on a review of the following hierarchy of servicing alternatives:

- The potential of extending local municipal sewer systems or of pumping the sewage to a local municipal treatment system;
- The potential to expand existing communal sewage treatment facilities in the area and to service the development via these facilities;
- The potential of developing local communal sewage treatment facilities to service the proposed development;
- The use of an on-site sewage treatment system; and
- The availability of municipal sewage treatment facilities or other facilities that can legally accept septage from LSSDS.

22.3 EVALUATION OF SITE CHARACTERISTICS

The proposed site for a LSSDS needs to be evaluated more rigorously than a single-residence site because of the larger volume of sewage that is to be applied and the greater need to determine hydraulic gradients of the groundwater. The designer needs to ensure that a system that applies a larger hydraulic load to the subsurface over a greater area does not exceed the site's capacity to accept it. Restrictive soil horizons that may inhibit deep percolation need to be identified before proceeding with design. Groundwater mounding analysis should be performed to determine whether the hydraulic loading to the saturated zone, rather than the loading to the infiltration surface, controls system sizing.

The site should be evaluated for conditions that might inhibit construction or proper operation of the LSSDS. With input from the groundwater specialist, the designer should predict the direction of groundwater flow and propose the most logical location for a drainfield/dispersal field. In most cases a large drainfield/dispersal system will need a large amount of owner controlled, downgradient area to fully treat and dilute the effluent. It is recommended that there be no on-site wells, existing or planned, downgradient of the dispersal field, and that the field and any on-site wells, existing or planned, be located such that the wells would not draw in water that is affected by sewage.

Siting of the LSSDS should be done as early as possible in the land development planning process such that the hydrogeological assessment can be used to determine the appropriate drainfield/dispersal site, able to treat and dilute the effluent, before the housing locations are established. A decision has to be made early on as to whether the housing development is going to be serviced by individual drinking water wells or a communal drinking water system. Locating houses with individual drinking water wells downgradient from LSSDS drainfields is not advised unless it can be shown that the situation is protective of drinking water quality. The designer should choose a potential dispersal system location that best accommodates the above considerations based on the predicted groundwater flow direction, existing drinking water supplies and the plan for future drinking water supply.

22.4 SOIL EVALUATION

The site evaluation should include digging test pits or drilling boreholes at the proposed drainfield location to reveal soil and groundwater conditions. Soil conditions should be properly evaluated before designing the drainfield.

The purpose of the soil evaluation is to:

- Determine the depth to the primary restricting layer (high groundwater level, impermeable soil or rock). At least 0.9 m (3.0 ft) separation distance is recommended. Issues regarding depth to rock may also arise relative to possible undesirable effects on water resources (*Section 22.5 - Assessment of Impact on Water Resources*);
- Determine the influence of any secondary restrictions (hard pans, abrupt textural changes, disturbed soil);
- Determine the infiltration rates;
- Determine the ability of soil to transmit the effluent from infiltration surface to deeper and distant layers (percolation/linear loading rate/groundwater mounding);
- Assess treatment abilities of the unsaturated soil (based on soil texture and structure). Preliminary assessment criteria are provided in Table 22-1; and
- Determine any construction related concerns (smearing, compaction).

Table 22-1 - Suggested Hydraulic and Organic Loading Rates for Sizing Infiltration Surfaces

Texture	Structure		Hydraulic loading ($L/m^2 \cdot day$) ($gallons/ft^2 \cdot day$)		Organic loading ($g\ BOD_5/1000m^2$-day) ($lb\ BOD_5/1000ft^2$-day)	
	Shape	Grade	BOD_5=150 mg/L	$CBOD_5$=30 mg/L	BOD_5=150 mg/L	$CBOD_5$=30 mg/L
Coarse sand, sand, loamy coarse sand, loamy sand	Single grain	Structureless	32 (0.8)	65 (1.6)	4880 (1)	1950 (0.4)
Fine sand, very fine sand, loamy fine sand, loamy very fine sand	Single grain	Structureless	16 (0.4)	40 (1)	2440 (0.5)	1220 (0.25)
Coarse sand loam, sand loam	Massive	Structureless	8 (0.2)	24 (0.6)	1220 (0.25)	730 (0.15)
	Platy	Weak	8 (0.2)	20 (0.5)	1220 (0.25)	630 (0.13)
	Prismatic, blocky, granular	Weak	16 (0.4)	28 (0.7)	2440 (0.5)	880 (0.18)
		Moderate, Strong	24 (0.6)	40 (1.0)	3661 (0.75)	1220 (0.25)
Fine sandy loam, very fine sandy loam	Massive	Structureless	8 (0.2)	20 (0.5)	1220 (0.25)	634 (0.13)
	Prismatic, blocky, granular	Weak	8 (0.2)	24 (0.6)	1220 (0.25)	730 (0.15)
		Moderate, Strong	16 (0.4)	32 (0.8)	2440 (0.5)	980 (0.2)
Loam	Massive	Structureless	8 (0.2)	20 (0.5)	1220 (0.25)	630 (0.13)
	Prismatic, blocky, granular	Weak	16 (0.4)	24 (0.6)	2440 (0.5)	730 (0.15)
		Moderate, Strong	24 (0.6)	32 (0.8)	3660 (0.75)	980 (0.2)
Silt loam	Massive	Structureless		8 (0.2)	0.00 (0)	240 (0.05)
	Prismatic, blocky, granular	Weak	16 (0.4)	24 (0.6)	2440 (0.5)	730 (0.15)
		Moderate, Strong	24 (0.6)	32 (0.8)	3660 (0.75)	980 (0.2)
Sandy clay loam, clay loam, silt clay loam	Massive	Structureless				
	Platy	Weak, mod., strong				
	Prismatic, blocky, granular	Weak	8 (0.2)	12 (0.3)	1220 (0.25)	390 (0.08)
		Moderate, Strong	16 (0.4)	24 (0.6)	2440 (0.5)	730 (0.15)
Sandy clay, clay, silty clay	Massive	Structureless				
	Platy	Weak, mod., strong				
	Prismatic, blocky, granular	Weak				
		Moderate, Strong	8 (0.2)	12 (0.3)	1220 (0.25)	390 (0.08)

Source: Adapted from Tyler, 2000.

Evaluation of soil should be performed by a soils professional. The first step in a soils evaluation involves reviewing the applicable soil survey information. The next step involves the examination of the soil in soil pits. Soil descriptions should be recorded. Enough test pits should be excavated and analyzed to adequately characterize the site and should be located within or near the system boundaries. It is important to locate test pits such that the disturbed soil will not interfere with the future absorption area. In addition samples from deep borings to a depth of 3 m (10 ft) or more below the water table should be evaluated to determine the hydraulic capabilities of these deeper materials. Soil description including soil texture, structure and colour from the test pits should be part of the design documentation.

22.5 ASSESSMENT OF IMPACT ON WATER RESOURCES

Developments in the science of contaminant hydrogeology and in environmental regulations and policies may require changes in the ministry's approach to water resources impact assessments. Also, certain aspects of the prediction of effects on water resources are highly case-specific and site-specific. Therefore, pre-submission consultation with ministry groundwater and surface water staff at the local Regional Office is strongly advised.

The following guidance for water resource impact assessments uses concepts presented in ministry *Guideline B-7, Incorporation of the Reasonable Use Concept into MOEE Groundwater Management Activities* and *Procedure B-7-1, Determination of Contaminant Limits and Attenuation Zones*. These are commonly referred to as the "*Reasonable Use Guideline*". While the guidance presented here is partly based on the *Reasonable Use Guideline*, an assessment performed in support of an application for approval of a LSSDS should refer primarily to the guidance given in the *Section 22.5 - Assessment of Impact on Water Resources*.

This guidance applies to those sewage systems which discharge to the subsurface and are governed by the requirements of Section 53 of the *Ontario Water Resources Act*, including sewage spray irrigation systems.

A water resources impact assessment is required to assess the risk of undesirable effects of the sewage, from the point where it enters the subsurface, on surrounding water bodies, water resources, and other users, including all groundwater and surface water that may be significantly affected. The focus of such an assessment is on the effect of the sewage constituents on the quality of waters relative to any function or use, potential or actual, of those waters. This assessment should take into account the design of a sewage works, especially as the design would affect effluent quality. In turn, the design of the works would need to minimize the risk of undesirable environmental effects.

Since the groundwater regime is the initial medium to receive sewage effluent, the primary technical review for these assessments will be done by a hydrogeologist in the ministry Technical Support Section at the local Regional Office. The degree of detail required for the assessment of surface water body effects is especially dependant on the attributes of receiving water bodies. Therefore, pre-submission consultation with ministry groundwater and surface water staff regarding potential surface water impacts is advised.

22.5.1 Ministry Documents

Other Ministry documents relevant to LSSDS approvals are:

- *Guideline B-1 Water Management - Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of the Environment*;
- *Guideline B-7 Incorporation of the Reasonable Use Concept into MOEE Groundwater Management Activities*;
- *Procedure B-7-1 Determination of Contaminant Limits and Attenuation Zones*;
- *Wells Regulation Ontario Regulation 903, Revised Regulations of Ontario 1990*;
- *Clean Water Act*; and
- *Authorship of Water Resource Impact Assessment*.

22.5.2 Authorship of Water Resource Impact Assessment

Technical submissions need to be prepared by a geoscientist, designated P.Eng. or P.Geo., qualified to prepare assessments of groundwater quality impacts and, where necessary, by persons qualified to prepare surface water quality impact assessments.

Technical submissions should contain the names, signatures, and qualifications/designations of the authors.

22.5.3 Pre-submission Consultation and Preliminary Groundwater Impact Assessment

For new developments, a proponent should undertake a preliminary assessment, utilizing existing data, to determine feasibility. Usually a proponent's property can support some level of development relying upon commercially available on-site sewage treatment technology. The preliminary assessment should evaluate the subject property and the surrounding properties to determine what level of development can take place within the bounds of ministry requirements and an appropriate level of risk.

The preliminary assessment should be presented to Water Resources staff at the ministry's Regional Office as part of pre-submission consultation. These preliminary steps are for the proponent's benefit to avoid the expenditure of large sums of money in advancing a proposal that may be infeasible or uneconomical because of stringent effluent discharge criteria necessitated by the hydrogeological environment or the sensitivity of receiving surface waters.

Also in the case of a simple replacement or minor alteration of an infiltration facility, the proponent should contact the ministry to determine what degree of evaluation may be required. (Refer to *Section 22.5.18 - Replacement of Infiltration Facilities*.)

22.5.4 Scope of Detailed Water Resource Impact Assessment

In most cases a detailed groundwater impact assessment should be completed to adequately characterize the site and determine the anticipated impact of the development upon the environment and public health. Recommendations for the scope of a detailed groundwater impact assessment are outlined in *Section 22.6 - Scope of Detailed Water Resources Impact Assessment in the Design of LSSDS*.

22.5.5 Critical Contaminants

Most proposals for LSSDS involve domestic sewage. In the case of groundwater, the critical contaminant is typically nitrate. It should be assumed that all nitrite and ammonia will convert to nitrate. All water quality assessments should report the concentrations of these substances in units of "as Nitrogen" such as, "nitrate-N", "nitrite-N" and "total ammonia-N".

In the case of surface waters, the critical contaminants are usually ammonia and phosphorus. The ministry is proposing to establish an objective for nitrate in surface water. Should that be finalized, nitrate may become a critical contaminant for surface water too.

If the system is treating any other waste (e.g. industrial or commercial waste), or contains quantities of non-domestic wastes in excess of those typically found in domestic wastes, it is the responsibility of the proponent to evaluate the effluent quality in order to identify the critical contaminant(s).

22.5.6 Water Quality Limits

With respect to groundwater, the impact of a sewage system using subsurface discharge is assessed using a procedure derived from the ministry's *Guideline B-7, Incorporation of the Reasonable Use Concept into MOEE Groundwater Management Activities*. It is generally assumed that the reasonable use of any groundwater is drinking water. Consequently, the allowable contaminant concentration limits in groundwater at a property boundary are normally based on relevant drinking water quality standards such as the Ontario Drinking Water Quality Standards (ODWQS).

Water quality limits for the purposes of water resources impact assessment are a fraction of the relevant drinking water standards. The maximum allowable concentration at the property boundary for a substance that originates in the sewage and that has an applicable drinking water quality limit, such as an ODWQS, is one quarter of a health-related limit and one half of an aesthetic limit. In a situation where the reasonable use of groundwater is drinking water, the maximum concentration for nitrate in groundwater affected by sewage effluent is 2.5 mg/L as N because the health-related ODWQS for nitrate is 10 mg/L as N. The equivalent concentrations for chloride, which is an aesthetic parameter, are 125 and 250 mg/L, respectively. Note that background concentrations are not used in the calculation of allowable concentrations for the purposes of the water resources impact assessment.

However, determining background concentrations is normally necessary for monitoring purposes. (Refer to *Section 22.5.7 - Existing and Background Concentrations* and *Section 22.5.8 - Prediction of Contaminant Attenuation* for more information.)

Where the plume originating at the sewage system discharges to surface waters, whether these waters are on or off site, the assessment of impact would use procedures derived from *Guideline B-1 Water Management - Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of the Environment*. Although the Provincial Water Quality Objectives would be used in such assessments, the determinations of water quality limits are not as straightforward as for the drinking water case. The determination of limits is much more dependant on receiver-specific characteristics, and the ¼ and ½ fractions used in the drinking water case do not apply. Phosphorus and ammonia are likely to be contaminants of interest in the surface water case.

In certain cases, the applicant may wish to propose other reasonable uses or other water quality standards, such as where activities in the downgradient area are not and will not in the future be dependant on groundwater or on a particular use of groundwater. Discretion to accept or reject any such proposals lies with the ministry's Regional Director. Very early pre-submission consultation is advised in such cases.

22.5.7 Existing and Background Concentrations

Existing and background concentrations of critical contaminants should generally be used only for reference, not for calculation of allowable water quality limits. Obtaining and analyzing samples and recording the concentrations is normally required as part of the impact assessment and may also be required as part of a long-term monitoring program for the purposes of differentiating between the effects of the subject sewage system and other sources. Groundwater sampling to determine existing background concentrations should be undertaken using monitors located on the site and with screened intervals within the hydrostratigraphic unit that will receive the effluent.

22.5.8 Prediction of Contaminant Attenuation

For the basic calculation of impact of effluent on groundwater quality at a receiver (i.e., a property boundary or a surface water body), a constant quantity of dilution, which is a surrogate for all attenuative mechanisms, should be used in most cases. That quantity is 250 mm (10 in) of water per year over the area of the contaminant plume, which is approximately the rate of infiltration of precipitation into a sand unit. It has been used as the amount of uncontaminated water that mixes with and dilutes a contaminant plume in a sand environment. Since soils with a finer texture are likely to be more attenuative than sand, but would likely allow less infiltration, the sand-related quantity is applied as a surrogate for all attenuative mechanisms to all soil textures.

In the case of a leaching bed, the area of the plume is the product of the width of the leaching bed transverse to the groundwater gradient and the distance from the upgradient edge of the leaching bed to the relevant receptor (that is, the leaching bed area is included in the plume area). For an exfiltration lagoon, the dimensions are similarly obtained from the footprint of the exfiltration area and the downgradient area. For a spray irrigation system, it is the dimensions of the area of spray application and the downgradient area.

The area of the plume depends on the width of the area of sewage infiltration, and the greater this width, the greater the dispersion of the sewage input to groundwater. Increasing the width of the sewage infiltration area and increasing the distance to the receptor of interest would both contribute to increasing attenuation.

The area/dilution approach described above may not always be appropriate, such as where effects on surface water are being considered, or where soils are of low permeability, or where the nature of the effluent differs significantly from typical household wastewater. Please refer to *Section 22.5.11 - Attenuation of Phosphorus and Ammonia* and to *Section 22.5.14 - Low Permeability Environments*. A basic groundwater dilution schematic and formulation is provided below.

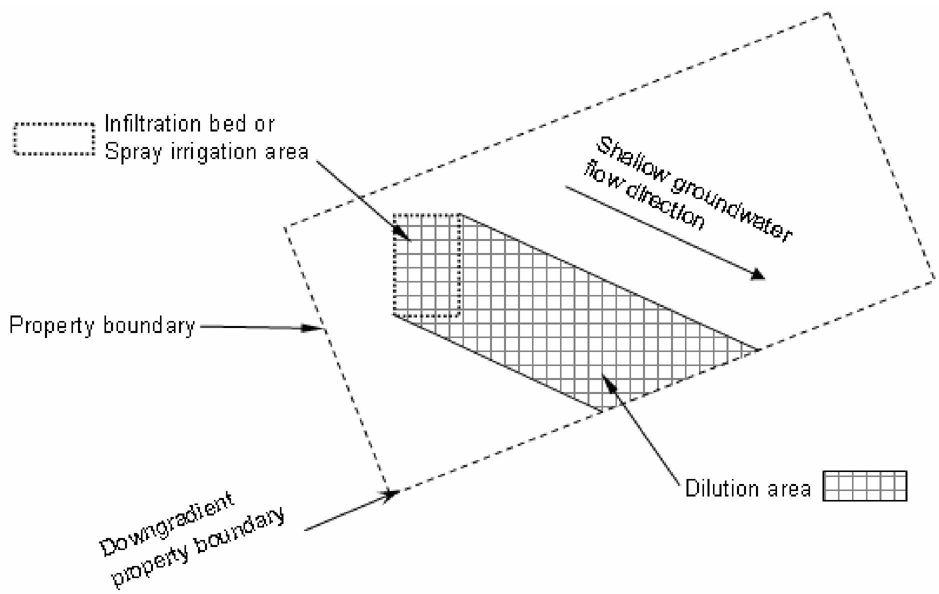

Figure 22-1 - Sample schematic for a basic groundwater dilution calculation

With reference to Figure 22.1, estimates of the annual dilution volume (V_A), the total volume of water (V_T) and concentration at the property boundary (C_{PB}) may be calculated as follows:

$$V_A = A_D \times k$$

$$V_T = V_A + V_S$$

$$C_{PB} = \frac{C_S \times V_S}{V_T}$$

where:

V_A = annual dilution volume [m^3 (US gal)];
A_D = dilution area [m^2 (ft^2)];
V_T = total volume of water [m^3 (US gal)];
V_S = annual sewage volume [m^3 (US gal)];
C_{PB} = concentration at property boundary [mg/L (lb/US gal)];
C_S = concentration in sewage [mg/L (lb/US gal)]; and
k = 0.25 m (SI) or 6.23 US gal/ft^2 (US).

The above calculation assumes a 250 mm (10 in) annual dilution precipitation rate (k).

22.5.9 Sewage Effluent Volumes

Sewage system design flows normally incorporate a safety margin and may therefore be greater than actual flows. A realistic estimate of the daily flows should be based on monitoring of similar, existing systems or, in the absence of such performance data, on accepted facility-based standards. In this case, a *Certificate of Approval* (C of A) granted by the ministry would stipulate a maximum flow and may require a program of monitoring of flows to ensure that the assessment assumptions are valid.

In the case of systems that are only occasionally used, such as seasonal-use facilities, it is assumed that the contaminant plume moves as a slug with advecting groundwater and that it affects water quality when it arrives at the receptor in the same way that a continuous source would. Therefore, an occasional-use system should be assessed as though it was a continuous-use system.

22.5.10 Effluent Quality

The proponent needs to determine the concentration of the contaminants of concern in the sewage effluent at the point of input to the infiltration surface. The proponent should acquire such information from documented monitoring of similar facilities. Where such information is not available, and sewage is expected to be a typical domestic type, an estimated nitrate-nitrogen concentration of 40 mg/L is generally acceptable.

Where a treatment facility is required to reduce the concentration of a contaminant in the sewage effluent to a specified level before discharge to the subsurface, documentation of the treatment system performance is required. In this case, monitoring of effluent quality would be needed. Contingency plans in case of failure may also be necessary, and would become conditions in a C of A.

22.5.11 Attenuation of Phosphorus and Ammonia

Phosphorus is of concern relative to surface water. Many geological materials have a high capacity to attenuate phosphorus by precipitation in the unsaturated zone and by adsorption below the water table. However, the attenuative capacity of a geological material is limited, and often reversible, which can result in significant phosphorus movement over the long term. In some settings, such as those with exposed fractured bedrock or with only a thin veneer of overburden, the phosphorus attenuation capacity may be quite low.

In the case of proposed sewage systems on the Precambrian Shield, the effects of phosphorus discharge to surface waters should be evaluated considering the requirements and recommendations in the document *Lakeshore Capacity Assessment Handbook - Protecting Water Quality in Inland Lakes on Ontario's Precambrian Shield - Consultation Draft*, December 2007 or its replacement.

Un-ionized ammonia (NH_3) is also a concern. In cases where anaerobic conditions are maintained between the sewage system and the surface water body, ammonium may not be converted to nitrate before it discharges to surface water.

The potential for surface water impact increases as the distance to the point of plume discharge to the surface water decreases. In most cases, a separation distance of 300 metres (980 feet) between the area of sewage infiltration and the surface water body should be sufficient to ensure that there are no appreciable effects to surface water quality. However in certain situations such as where there are particularly sensitive receivers or where different surface water quality standards may apply, an assessment would be required.

Where the surface water body is relatively shallow, the assessment may need to include a three-dimensional evaluation of groundwater flow patterns to determine whether the plume actually discharges to the surface water body.

22.5.12 Microbial Pathogens

The risk to human health from pathogens originating in a sewage system is greater where the effluent has access to an aquifer with high groundwater flow velocities. This would occur where the aquifer is exposed at surface and consists of coarsely textured granular overburden or fractured or karstic bedrock, or where the soil overlying an aquifer is a fine-textured soil of limited thickness penetrated by root holes, desiccation cracks or other highly conductive features. In such groundwater settings, the potential movement of pathogens is to be taken into consideration in evaluating the adequacy of the setback from the downgradient property boundary. Monitoring for specific pathogens or indicator organisms may be required in settings where rapid off-site movement of pathogens in groundwater is possible.

The maximum acceptable microbial pathogen content in drinking water is nil. In order to ensure adequate attenuation, the subsurface travel time between the area of sewage infiltration and the boundary or receptor should be considered. Where higher velocities are likely, treatment to remove/inactivate human pathogens may be required.

22.5.13 Assessment of Impact in Shallow Bedrock Environments

Shallow bedrock environments often pose difficulties both with respect to the attenuation of sewage-derived contaminants and the characterization of groundwater flow. Some of the problems encountered in these environments are:

- Unpredictable flow patterns governed by fracture/bedding plane orientation and geometry;
- High groundwater flow velocities; and
- Low attenuation capabilities.

In shallow bedrock environments, the additional risk will need to be taken into account and special construction (e.g. additional material in constructed leaching beds) or additional assurances regarding effluent quality may be required to ensure compliance with relevant criteria. The higher groundwater flow velocity with increased pathogen survival rate may also require provision for treatment to remove/inactivate human pathogens.

22.5.14 Low Permeability Environments

Where it can be shown that the uppermost subsurface unit(s) at an infiltration facility have a vertical hydraulic conductivity of 10^{-5} cm/sec or less, is at least 10 metres (33 feet) thick and extends at least 100 m (330 ft) downgradient of the infiltration area, attenuation calculations may not be required. The assessment would however need to demonstrate the absence of higher permeability pathways in the lower permeability material.

In the case of a leaching bed, where there is such a thick, extensive and low permeability unit at surface, a raised bed may be needed. In the case of a sewage lagoon that will have a constructed or artificial liner designed to prevent leakage into the subsurface, the requirement for a water resources impact assessment may not apply. However, where a sewage lagoon is excavated into native soils, the risk of leakage may be sufficient to warrant a water resource assessment pursuant to this guideline. A proponent should engage the ministry Regional Office in pre-submission consultation at an early stage to address this issue.

22.5.15 Existing Subsurface Disposal Systems

Where there is an existing subsurface disposal system at a site, or at a nearby site in a sufficiently similar environment, and the plume from that system has had sufficient time to reach a stable condition on site, the assessment of the existing impact from that system would need to be used as part of the water resources impact assessment for the proposed system. Extrapolation from actual performance data is considered to be a more reliable approach than a prediction based on theoretical assumptions that are difficult to verify. The extent to which a water resources impact assessment can rely upon an assessment of existing systems will largely depend on the similarity between the existing and proposed systems and environments. This should be evaluated on a case-by-case basis.

22.5.16 Contaminant Attenuation Zones

Where there is insufficient distance between the infiltration facility and the downgradient property boundary to meet required water quality limits at a property boundary, the proponent may wish to negotiate a formal Contaminant Attenuation Zone (CAZ) or other instrument with the owner(s) of the downgradient property(s) to acquire sufficient area. A CAZ would need to be agreed among other parties such as the municipality and the ministry, and it would have to be written into documentation of land ownership and land use planning. The ministry will require that the owner of the property containing the CAZ register a Certificate of Requirement in the Land Registry. The Certificate of Requirement notifies anyone with an interest in the property of the presence of the CAZ.

22.5.17 Assessment of Impact on On-site Wells

In the case of groundwater, the primary focus of the water resources assessment is on off-site impacts. The potential for degradation of on-site well water is also a concern and needs to be assessed. Any on-site wells, whether in use or not, should be located and examined for contamination potential. On-site wells and other subsurface structures may act as a conduit for vertical contaminant movement. The location of infiltration area should be chosen to ensure that the quality of water from on-site wells is not compromised.

On-site wells need to be constructed, maintained and, if unused or subject to contamination, abandoned in accordance with O. Reg. 903.

The water resource impact assessment should:

- Identify all on-site water wells, whether in use or not, including information from Water Well Records, consultants' reports and hearsay;
- Specify if the on-site water wells are or might be used as a source of drinking water;

- Review the location of the on-site water wells with respect to separation distances between wells and contaminant sources, as may be required by the Part 8 of Division B of the *Building Code* (O. Reg. 350/06) and *Wells Regulation* (O. Reg. 903);
- Assess the construction and integrity of the on-site water wells with respect to the requirements of O. Reg. 903;
- Determine the presence of any abandoned oil wells;
- Assess the potential for the well to be a conduit for contaminant movement; and
- Assess the potential for contamination of on-site water wells.

Where there is significant potential for contamination of an on-site well that is or may be used for human consumption, or where the well may act as a conduit for contaminant migration, proper abandonment of the well may be required as a condition of the C of A.

22.5.18 Replacement of Infiltration Facilities

In some cases where an existing subsurface sewage disposal system is being replaced and there is no increase in design capacity, a detailed water resources impact assessment may not be required. However, in some replacement scenarios, such as where there are incidents or complaints of contamination in any on-site or off-site wells or in groundwater or surface water, some form of assessment would likely be required. Pre-submission consultation with the Ministry's Regional Office would be essential in such cases.

Proponents of replacement systems are encouraged to optimize the available on-site attenuation capacity of the site and apply reasonable technology in the replacement system design to reduce water resources impacts. Monitoring requirements for replacement sewage infiltration works should be evaluated on a case-by-case basis.

22.5.19 Post-Construction Monitoring and Contingency Programs

The water resource impact assessment should address the need for groundwater and surface water monitoring and contingency planning. These activities would be required in environments in which plume behaviour would be particularly difficult to characterize or where the consequences of failure of the predictions are of particular concern.

Circumstances in which a comprehensive monitoring program would likely be needed include the presence of:

- High groundwater flow velocities;
- Low attenuation capabilities;
- Specific effluent quality requirements; and
- Proximal receptors.

The components that may be required in a comprehensive monitoring program include:

- Clearly stated goals and objectives;
- A schedule for the monitoring of flows and quality of sewage at the point of input to the infiltrative surface;
- A plan for the location of monitoring wells in the plume, with provisions for future determination of whether the plume is delineated and whether additional wells are necessary;
- The designation of other wells and other water bodies for monitoring;
- A schedule for water level and water quality monitoring in wells and water bodies, including identification of the contaminants to be monitored;
- A schedule for continual assessment and reporting on compliance and efficacy of the program;
- A contingency plan for dealing with any problems that arise or that may reasonably be predicted to arise and a commitment to mitigate undesirable impacts;
- A protocol for routine and extraordinary reporting to the ministry; and
- A schedule of regular maintenance of the treatment works.

The monitoring reports prepared for the sewage works should present the results of the monitoring and maintenance in a manner that facilitates regulatory review. Monitoring reports should include, but not be limited to:

- Location and site maps (to scale) showing relevant features;
- Plume maps for important parameters, in three dimensions where necessary;
- Piezometric maps with interpreted flow directions and data points shown;
- Tabular summaries of data;
- Appended laboratory reports;
- Quality Assurance and Quality Control assessment of data and sample collection;
- Interpretation of attenuation processes within the plume and development of the plume relative to predictions made in the water resources impact assessment;
- Assessment of compliance with the relevant water quality criteria for groundwater and surface water and effluent quality criteria of sewage treatment processes, if any; and
- Recommendations for remedial action and changes to the monitoring program, including consideration of whether the plume is being adequately delineated and whether an expansion or other improvements in the monitoring program is needed.

The frequency of monitoring report submission should normally be less than annually, but will be decided on a case-by-case basis. More frequent submission may be required at the beginning of site monitoring or system operation. In cases where the actual submission of reports to the ministry is not required, the ministry may nevertheless require the owner of the system to produce and store routine reports and provide them to ministry staff on request or to send status letters confirming that required monitoring has been completed.

22.5.20 Geological Information in Support of Sewage Works Design

The Part 8 of Division B of the *Building Code* (O. Reg. 350/06) sets standards for small sewage systems and contains some design requirements that may also apply to LSDDS. The Code sets the range of the soil percolation time where a leaching bed could be constructed and includes design requirements for:

- Vertical separation from the high water table; and
- Vertical separation from bedrock or soils of low permeability.

The above requirements are to be considered and addressed as part of the overall system design with respect to LSSDS. The requirements in the *Building Code* represent the minimum standards acceptable for the installation of small systems (10,000 L/d or less). These standards may need to be increased for LSSDS and the designer of the LSSDS should assess the suitability of the use of these minimum standards for site-specific conditions.

22.6 SCOPE OF DETAILED WATER RESOURCES IMPACT ASSESSMENT IN THE DESIGN OF LSSDS

The following subsections provide bulleted lists outlining the scope of design information to be considered and included when conducting a detailed impact assessment for LSSDS designs.

22.6.1 Field Activities and Data Collection

A detailed water resources impact assessment may need to contain the following site-specific field work and data collection activities as well as any other information that prove to be necessary:

- Review of available Water Well Records, source water protection mapping etc., topographic maps and geological maps;
- Inspection of the site and its immediate vicinity with respect to land use, topography and vegetative cover as these might affect infiltration;
- Test pits, boreholes and associated logs;
- Examination of nearby road cuts, banks, erosional features, or excavations;
- Monitoring well installation;

- Hydraulic conductivity testing;
- Soil sampling and grain size analysis;
- Groundwater sampling and analysis for parameters including the critical contaminant (e.g. nitrate and nitrite) and other key contaminant indicator parameters (e.g. nitrate plus nitrite plus ammonia, electrical conductivity, dissolved oxygen, TOC, pH, phosphorus, TKN, sodium, chloride, metals, and potassium);
- Determination of static water elevations in wells;
- Inspection of the site and its immediate vicinity for evidence of permanent, intermittent or ephemeral water courses;
- Determination of relative elevations of surface water bodies on-site or in the immediate site vicinity; and
- Door-to-door survey in the site vicinity to determine water well use and characteristics.

22.6.2 Data Interpretation and Presentation

Interpretation and presentation of the data should include:

- Description of the methodology used to determine monitoring well elevations;
- Potentiometric plan maps with indications of groundwater flow directions;
- Presentation of hydrogeological information on cross-sections across the site and site vicinity (both parallel and transverse to the main groundwater flow direction);
- Summary tables of analytical data;
- Establishment of natural background and existing background levels of the critical contaminant(s);
- Determination of Reasonable Use of groundwater on the adjacent property; and
- Description of the expected lateral, vertical and longitudinal configuration of the plume.

22.6.3 Appended Information

Supporting information appended to the Groundwater Impact Assessment should include:

- Borehole/monitoring well logs;
- Door-to-door water well survey results;
- Laboratory analytical reports;
- Field analytical data;
- Confirmation from the analytical laboratory that it is certified for the analyses performed;
- Copies of water well records used in the assessment;

- Copies of permits for water takings in the site vicinity;
- Historical measurements of static well water elevations;
- Decommissioning procedures, proposed or already implemented, for boreholes, monitoring wells and test pits used in the investigation;
- Field notes for survey work done to establish monitoring well elevations or the report of a licensed surveyor; and
- Hydrogeology studies done in support of nearby development, which may be available in municipal planning offices.

22.6.4 Available Information

Information that should be retained by the consultant but available to the ministry upon request includes:

- Data collected during well development (as distinct from well purging) to demonstrate that the well is properly developed to enable representative groundwater samples to be obtained.

22.7 DESIGN CONSIDERATIONS

22.7.1 LSSDS Versus Small Subsurface Sewage Disposal Systems

The principal distinction between a LSSDS subject to OWRA and small on-site sewage disposal systems subject to Part 8 of Division B of the *Building Code* (O. Reg. 350/06) standards lies in the area of complexity and the need for professional engineering design and construction supervision. A sewage system servicing a large development may have a much higher daily sewage flow, variations in peaking factors, and changing characteristics of the sewage. Similarly, it could service a large residential development and have similar sewage characteristics but require a sizeable collection system involving sanitary sewers, manholes and pumping stations. These types of systems need to be designed with consideration for site topography, hydraulic capacity, surface drainage, groundwater movement, and the overall impacts on the surrounding environment.

Every application for a large subsurface disposal system should be evaluated individually for specific site constraints. More extensive background studies may be required in support of a LSSDS to ensure compatibility with the site and protection of the environment. These may include, but are not limited to:

- Hydraulic dispersal to the soil of large volumes of sewage effluent may produce problems out of proportion to the increase of flows from a small to a large system. This is especially true where the soils underlying the drainfield present an increasing resistance to downward percolation as the depth increases, and where the lateral outflow potential of the more permeable upper soil layers limits site acceptability. The resistance to dispersal in the soil causes the sewage effluent to mound over the area of its application to a height that will create sufficient pressure to overcome the resistance. Dispersal in the underlying and surrounding soil without breakout to the surface is affected by:

- The infiltration area covered by the drainfield. For the same sewage flow, a leaching bed constructed in a soil of low percolation time will require less area and will concentrate the application of sewage effluent to the soil per unit of area, compared to a leaching bed treating the same sewage flow in a soil of higher percolation time;
- The permeability and thickness of the underlying soil strata;
- The depth to water table and its hydraulic gradient. The possibility of peak sewage flows occurring at the same time as high groundwater conditions, and the effect of heavy lawn sprinkling, or the diversion of surface waters, should be considered;
- The direction of movement of the groundwater away from the bed, and whether or not the subsurface configuration at some point in that direction may restrict the outflow of sewage;

- The discharge in one location of a relatively large amount of contaminants into the soil makes their attenuation more difficult than in individual residential systems and emphasizes the need to assess the effects on the groundwater;
- The sewage collection system may be much more extensive than a single building sewer and include manholes, lift stations, pump chambers, and other structures;
- The assessment and computation of daily sewage flow requires knowledge of peak hourly and daily flows necessary for the proper selection and design of a treatment plant to meet these conditions;
- The site conditions such as site topography, drainage, high water table and the prevalence of rock or soils of low to unacceptable permeability may be restrictive to the location of large sewage systems because of the large area required. Where fissured rock is prevalent in the area, the location may be unacceptable because of the adverse effects on groundwater; and
- In large commercial systems, some constituents in the sewage, may be present in greater proportions than they are in residential sewage, and thus have a greater bearing on equipment selection and system design. An example is the concentration of washing detergents or disinfectants. Wastewater with a significant heavy metals content is not suited for treatment in a LSSDS.

The designer of a LSSDS should consider the need for:

- A requirement to register on title the need for a service contract or the need for including information regarding the servicing requirements;

- Any long term maintenance or operation agreements that may be required;
- A requirement for a spare area to allow for system enlargement should problems arise;
- Designation of any area which is set aside for expansion of the bed to handle expansion of the development;
- Any specific requirements respecting construction, such as supervision, certification by the professional engineer, or completion of as-constructed drawings, operation or maintenance;
- Any special conditions with respect to ongoing monitoring of the system;
- Any source water protection requirements, where applicable;
- Any limitations on the type of sewage flow to be treated; and
- Any restrictions on the use of the property or neighbouring properties with respect to the attenuation zone of the effluent plume.

Notwithstanding the above, the designer of the LSSDS is advised to consider, where appropriate and applicable, the design standards for small subsurface disposal systems contained in Part 8, Division B, of the *Building Code*. However, clearances or separation distances from large systems to such features as wells, surface water bodies and property boundaries need to be determined on a site-specific basis - see *Section 22.5 - Assessment of Impact on Water Resources*.

22.7.2 Sewage Characteristics

If the quality of sewage is similar to that of typical domestic wastewater, then it may be reasonable to design the system for the same type of sewage treatment as a typical small system under Part 8, Division B of the *Building Code*. If the LSSDS is proposed to service dry industry, commercial facilities, institutional development, restaurants, office buildings or a larger residential development, it will be necessary to assess both the sewage quality and flow characteristics.

There are some types of wastewater that may not be suitable to be treated with a LSSDS. These may include wastewater from automatic car washes, garage facilities, or some agricultural uses such as egg washing. LSSDS for these types of sewage may require complicated pretreatment or this type of wastewater may not be suitable for subsurface disposal.

Typical characteristics of undiluted (no infiltration) residential sewage are presented in Table 22-2.

Table 22-2 Mass Loadings and Concentrations in Typical Residential Wastewater[1, 4]

Constituents	Mass Loading (g/person·d) (lb/person·d)	Concentration (mg/L)[2]
Total Solids (TS)	115 - 200 (0.253 - 0.44)	500 - 880
Volatile Solids (VS)	65 - 85 (0.143 - 0.187)	280 - 375
Total Suspended Solids (TSS)	35 - 75 (0.077 - 0.165)	155 - 330
Volatile Suspended Solids (VSS)	25 - 60 (0.050 - 0.132)	110 - 265
5-day Biochemical Oxygen Demand (BOD_5)	35 - 65 (0.077 - 0.143)	155 - 286
Chemical Oxygen Demand (COD)	115 - 150 (0.253 - 0.330)	500 - 600
Total Nitrogen (N)	6 - 17 (0.013 - 0.015)	26 - 75
Total Ammonia-Nitrogen (TAN)	1 - 3 (0.002 – 0.007)	4 - 13
Nitrites and Nitrates $(NO_2^- + NO_3^-) - N$	<1 (< 0.002)	<1
Total Phosphorus (TP)	1 - 2 (0.002 – 0.004)	6 - 12
Fats, Oils, and Grease	12 - 18 (0.026 - 0.040)	70 - 105
Volatile Organic Compounds (VOC)	0.02 – 0.07 (0.00004 -0.00015)	0.1 - 0.3
Surfactants	2 - 4 (0.004 – 0.009)	9 - 18
Total Coliforms[3]	-	$10^8 - 10^{10}$
Fecal **C**oliforms[3]	-	$10^6 - 10^8$

Notes:

1. For typical resident dwellings with standard water-suing fixtures and appliances.
2. Milligrams per litre, assuming wastewater generation rate of 225 litres/person/day (60 US gallons/person/day).
3. Coliforms concentrations presented in Most Probable Number of organisms per 100 millilitres.
4. Source: Adapted from Bauer et al. 1979; Bennet and Linstedt, 1975; Laak, 1975, 1986; Sedllak, 1991; Tchobangolous and Burton, 1991.

22.7.3 Design Sewage Flows

The computation of the design sewage flow for a large sewage system will vary according to the nature of the development to be served. Recommendations concerning design flows are given in *Section 5.5.2 – Design Sewage Flows*. Calculation of design peak daily and design peak hourly flows will be necessary to determine the requirements of all parts of the sewage system. Collection systems for LSSDS may consist of single building drains from sites such as schools, or may consist of manholes, gravity sewers, and sewage pumping stations if from multi-structured facilities such as commercial malls. These collection systems should be designed by professional engineers in accordance with municipal standards and ministry guidelines.

22.7.4 Pretreatment of Sewage

LSSDS performance is dependent on the efficiency of the pretreatment system (e.g. septic tank, aerobic treatment process, and/or filtration), the method of sewage effluent distribution and hydraulic and organic loadings to the soil infiltrative surface, and the properties of the vadose and saturated zones underlying the infiltrative surface.

The LSSDS is mainly comprised of two components, a pretreatment process(es) (i.e., a septic tank or other treatment processes facilities) followed by a soil component (e.g. drainfield). The pretreatment facilities should be designed to achieve the treated sewage effluent quality predetermined by the hydrogeological study and soil evaluation in the drainfield area. These quality requirements may include, in addition to $CBOD_5$ and TSS other parameters such as pathogens, nitrogen and phosphorus.

Sewage pretreatment processes that may be considered for LSSDS include: physical and biological such as anaerobic, aerobic, and anoxic. The basic design criteria for these processes have been discussed in preceding chapters of the Guidelines.

22.7.5 Septic Tanks

The septic tank is the most commonly used anaerobic pretreatment process for a small onsite sewage system for BOD_5 and TSS reduction (50 and 70 percent, respectively) and oil/grease removal through sedimentation and flotation. The designer of a LSSDS which is to utilize septic tank(s) should follow the construction details provided in Part 8 of the *Building Code*. The working capacity of the septic tank(s) should be at the minimum 24-hours retention at a design peak daily flow.

The tank geometry affects the septic tank efficiency. The length-to-width (L/W) ratio, surface area and liquid depth are important considerations. Compartmentalized, elongated tanks with L/W ratios of 3:1 and greater, reduce short-circuiting across the tank and improve suspended solids removal. Other elements in the design of the septic tank that should be considered by the designer include grease/oil interceptors, inlet and outlet devices, baffles, effluent filters, access openings, gas management and other issues.

22.7.6 Aerobic Processes

Using secondary aerobic biological treatment processes (other than primary septic tanks) for lowering concentrations of BOD_5 and TSS in the effluent is recommended. By lowering organic loadings in the sewage effluent, the drainfield required area may be reduced and the life of the LSSDS system prolonged.

For flows not substantially larger than 10,000 L/d the designer should consider the use of pre-engineered (package) aerobic biological treatment units.

22.7.7 Filtration

Sand or other media may be used in packed bed filters to provide advanced treatment to septic tank or secondary aerobic treatment unit effluent in intermittent (single pass) or recirculating operation modes. Foam blocks, wood chips, peat or coarse fibre materials are used in proprietary units which may provide additional process benefits such as nitrification and denitrification. The designer should refer to the manufacturer's specifications for further information.

Filtration of secondary treatment effluent is needed where application of a specific drainfield technology requires very high quality effluent. For a shallow buried trench method, used for soils with percolation time greater than 50 minutes, the effluent $CBOD_5$ concentration of 10 mg/L and TSS concentration of 10 mg/L should be consistently achieved.

22.7.8 Tertiary Treatment

The hydrogeological evaluation of the site and application of the ministry's Guideline B-7 guideline may result in the need for reduction of pathogens, nitrogen (nitrates) and/or phosphorus concentrations in the treated sewage effluent. For guidance on nitrogen removal the designer is referred to *Chapter 12 – Biological Treatment*, for guidance on phosphorus removal technologies to *Chapter 15 – Supplemental Treatment Processes* and for guidance on sewage disinfection to *Chapter 14 - Disinfection*.

22.7.9 Drainfield

The minimum drainfield infiltration surface area is a function of the maximum anticipated daily effluent volume to be applied and the maximum instantaneous and daily mass loading limitations of the infiltration surface. In sizing the infiltration surface, conservative infiltration loading rates are recommended. Morphologic features of the soil, particularly structure, texture, and consistence, are better predictors of the soil's hydraulic capacity than percolation rates. The infiltrative loading rates based on the soil morphology information shown in Table 22-1 may provide some guidance to the designers of LSSDS. The Table has two sets of loading rates; one for application of septic tank effluent (>150 mg/l BOD_5) and another set for secondary biological treatment process effluent (<30 mg/l $CBOD_5$).

In addition to sizing of the drainfield, the designer should give careful considerations of the placement, geometry, and depth of the infiltrative surface. Designers should consider the following for satisfactory long-term performance:

- Shallow placement of the infiltration surface;
- Trench orientation parallel to surface contours;
- Narrow trenches (width of < 900 mm [3 ft]);
- Timed dosing with peak flow storage;
- Uniform application of pretreated sewage over the infiltration surface; and
- Multiple cells to provide periodic resting, standby capacity, and space for future repairs or replacement.

The designer should be aware that the area included for calculation of dilution by infiltrating precipitation is the width of the drain field normal to the direction of groundwater flow multiplied by the length of the plume from the upgradient end of the drain field to the property boundary or surface water body. Therefore the designer should consider locating the drainfield as far upgradient of the property boundary or surface water body as possible and making the drainfield as wide as possible.

22.7.9.1 Placement of Infiltration Surface

Actual placement relative to the original soil profile at the site is determined by the desired separation distance between the infiltrative surface and the highest groundwater level, which may be below, at, or above the existing ground surface (in an in-ground trench, shallow buried, at grade, or elevated in a mound system).

Vertical separation between the infiltration surface and the water table needs to be maintained to achieve acceptable pollutant removals, sustain aerobic conditions in the subsoil, and provide an adequate hydraulic gradient across the infiltration zone. Treatment needs (performance requirements) establish the minimum separation distance, but in case of a potential for groundwater mounding, the separation distance should be appropriately increased. Effluent quality, hydraulic loading rates, temperature, soil characteristics, sewage effluent dosing pattern and distribution methods can affect the unsaturated soil depth needed for treatment. The designer should consider all these elements both separately and collectively when designing the system. Seeking reductions in vertical separation, or drainfield size based solely on the higher quality of the effluent being applied should be carefully considered, and combination of credits should be avoided. It should be noted that Part 8 of Division B of the *Building Code* requires 900 mm (3 ft) vertical separation between infiltration surface and a groundwater table for all code regulated systems regardless of the quality of the sewage effluent applied.

22.7.10 Depth of the Infiltration Surface

The depth of the infiltration surface is an important consideration in maintaining adequate subsoil aeration and frost protection in cold climates. The maximum depth should be limited to no more than 0.9 to 1.2 m (3 to 4 ft) below final grade to adequately re-aerate soil and satisfy the daily oxygen demand of the applied effluent.

22.7.11 Geometry, Orientation and Configuration of the Drainfield Infiltration Surface

The principles for design of large drainfields are similar to those of smaller systems for both raised and conventional beds but for large drainfields, more consideration should be given to the layout, configuration, and design of the drainfield. Layout and orientation will be critical as they will depend on the area which is available, the potential for groundwater mounding, the suitability of the subsurface for sewage disposal, and the impact on the groundwater. As an example, the drainfield should be located at the upgradient side of the site to allow for the maximum amount of dilution with the underlying groundwater.

The minimum setbacks to the drainfields may need to be increased for LSSDS. The clearances from wells, surface waters and property boundaries are to be established by the hydrogeological assessment rather than the minimum values set by Part 8 of Division B of the *Building Code*. Drainfields and any soil mantles (if required) should be kept an appropriate distance back from the top of any slopes or from areas that are potentially unstable. Minimum depths of the mantle and minimum clearances to groundwater should be increased to minimize the potential of groundwater contamination. Through the use of manifold chambers, flow levelers, and effluent pumping systems, a maximum amount of flexibility should be developed for effluent distribution within the drainfield. This allows the operator to manually adjust the system and respond to any type of breakout that may occur. The greatest concern with large drainfields is the ability to ensure even distribution of the effluent. The design recommendation on LSSDS sites are summarized in Table 22-3.

Table 22-3 - LSSDS Geometry, Orientation and Configuration

Absorption Trench	Design Considerations
Width	Preferably less than 0.9 m (3ft). Design width is affected by distribution method, constructability and available area.
Length	Restricted by available length parallel to site contour, distribution method and distribution network design.
Sidewall Height	Sidewalls are not considered an active infiltration surface. Minimum height is that needed to encase the distribution piping or to meet peak flow storage requirements.
Orientation/Configuration	Should be constructed parallel to site contours and/or water table or restrictive layer contours. Should not exceed the site's maximum linear hydraulic loading rate per unit of length. Spacing of multiple, parallel trenches is also limited by the construction method and slow dispersion from the trenches.
Bed	**Design Considerations**
Width	Should be as narrow as possible. Beds wider than 3.0 m to 5.0 m (10 to 16 ft) should be avoided.
Length	Restricted by available length parallel to site contour, distribution method, and distribution network design.
Sidewall Height	Sidewalls are not considered an active infiltration surface. Minimum height is that needed to encase the distribution piping or to meet peak flow storage requirements.
Orientation/Configuration	Should be constructed parallel to site contours and/or water table or restrictive layer or contours. The loading over the total projected width should not exceed the estimated down slope minimum liner hydraulic loading.

Source: Adapted from U.S. EPA – *Onsite Wastewater Treatment Systems Manual, EPA/625/R-00/008,* February 2002.

22.7.12 Effluent Distribution Onto the Infiltration Surface

The size of LSSD drainfield interface surface generally precludes the use of gravity flow to the drainfields. Part 8 of Division B of the *Building Code* mandates effluent distribution through dosing for any sewage system having more than 150 m (490 ft) length of distribution pipe. Typically, all LSSDSs fall within this category and should be dosed appropriately.

Pumps provide the most reliable method to alternately dose the drainfields. If the drainfields are not of equal sizes or are at significantly different elevations, it is possible to use different sized pumps set on a timer system to dose the drainfields. These timers should be set to dose a specific volume to each field and generally each field would need to be serviced by a separate effluent pump. The effluent is delivered to the distribution box and gravity is used to distribute sewage effluent in the laterals.

Pressure distribution relies on fully pressurizing the distribution pipes which are typically 34 mm (1.3 in) diameter with distribution holes 8 mm (0.3 in) in diameter). The combination of full pressurization, small pipe diameter and small perforations ensure that effluent is distributed equally along the lateral.

The design should evaluate the main means of dosing:

- Demand dosing;
- Timed dosing;
- Reduced dose volume;
- Orifices in the 12 o'clock or 6 o'clock position; and
- Network remaining full or partially full between doses.

A number of advantages and disadvantages are associated with each method and should be considered when designing dosing systems. These include:

- Demand dosing is the least complex system and therefore the least costly to install and operate. However, this system is not sensitive to heavy use days and/or hydraulic surges that overload the drainfield;
- Time dosing controls the discharge to the areas with evenly spaced doses. It allows for more frequent, smaller doses to the pumped and protects the receiving component from hydraulic overload. The system is however sensitive to heavy-use days and will activate alarms when volumes exceed design levels. The system is more costly and complicated than the demand dosing system, but can also help detect groundwater leaking into the septic tank or pump chamber;
- The reduced dose volumes method produces smaller more frequent doses with intervening resting and aeration periods, which assures unsaturated flow through the soil or filter media. These systems may require smaller orifices, pipes and valves that can increase the frequency of maintenance due to clogging;

- Orifices in the 12 o'clock position will maintain the laterals full or partially full and therefore reduce the amount of effluent required to pressurize the system. Maintaining effluent in the pipes and lines can promote biological growth which will cause clogging and a buildup of solids and slime that will require more frequent maintenance. Periodic draining of the laterals can reduce clogging problems, but will increase the dose volume required. Orifices in the 6 o'clock position will reduce clogging in the laterals being drained between dose cycles, but these systems will require a larger dose volume to pressurize the system; and
- Networks remaining full or partially full between doses have the same advantages and disadvantages as indicated for the other dosing methods: that is, more frequent doses with intervening resting and aeration will promote unsaturated flow through the system, but effluent lines that are full will promote biological growth.

APPENDIX I

GLOSSARY

Capacity

The flow rate that a treatment process unit or process train or treatment plant is capable of effectively processing.

Certificate of Approval

An authorization to initiate the establishment, construction or alteration of a sewage works including the right to operate, all in accordance with the *Ontario Water Resources Act R.S.O. 1990, c. O.40* , applicable regulations or conditions of approval. The Certificate of Approval (C of A) document consists of a detailed description of the works as well as terms and conditions of the approval.

Continuous monitoring equipment

Equipment that, at intervals appropriate for the process and parameter being monitored, automatically tests for the parameter directly in the stream (or in the case of UV application, through the stream) of water being treated or distributed, or in a continuous sample taken from the stream of water being treated or distributed, where a continuous sample is a continuous stream of water flowing from the stream of water being treated or distributed to the monitoring equipment.

Flux

For a membrane separation process, the volume or mass of permeate passing through the membrane per unit surface area per unit time. Solvent (water) flux is commonly expressed in gallons per square foot per day or cubic metres per square metre per second, or metres per second.

Groundwater

The water contained in interconnected pores located below the water table in an unconfined aquifer or in a confined aquifer.

Material Safety Data Sheet (MSDS)

Information on the use, handling and storage of specific chemicals or products. Material safety data sheets contain mandated types of information concerning physical characteristics, reactivity, required personal protective equipment and other safeguards.

Membrane backwashing

A cleaning operation that typically involves periodic reverse flow of permeate to remove particulate matter accumulated on the membrane surface; also referred to as backpulse, backpulse clean, or flux maintenance.

Ministry

Ontario Ministry of the Environment.

Net Positive Suction Head (NPSH)

A measure of the pressure at the suction side of the pump, including atmospheric pressure and vapour pressure of the liquid being pumped.

Permeate

For a pressure-driven membrane treatment process, the portion of the feed solution that passes through the membrane; known as “filtrate” for microfiltration.

Point of impingement

Any point on the ground or on a receptor, such as nearby buildings, located outside the emitter’s property boundaries at which the highest concentration of a contaminant caused by the aggregate emission of that contaminant from a facility is expected to occur.

Sewage Works

Works for the collection, transmission, treatment and disposal of sewage or any part of such works, but does not include plumbing to which the *Building Code Act, 1992* applies.

Surface water

Surface water bodies (lakes, wetlands and ponds, including dug-outs), water courses (rivers, streams, drainage ditches), infiltration trenches and areas of temporary precipitation ponding.

Water Works

Any part of a drinking-water system including collection, production, treatment, storage, supply and distribution of water or any part of such works.

Watershed

The drainage basin area contained within the bounds specified by a divide and above a specified point on a stream. A watershed is also called a catchment area, drainage area or drainage basin.

APPENDIX II

UNITS OF MEASURE

UNITS	DEFINITION
%	percent
^{o}C	degrees Celsius
^{o}F	degrees Fahrenheit
μm	micrometres or microns
ac	acres
atm	atmospheres
Btu/ft^3	British thermal units per cubic foot
Btu/lb	British thermal units per pound
cfm/1000 ft^3	cubic feet per minute per 1000 cubic feet
cfm/1000 US gal	cubic feet per minute per 1000 U.S. gallons
cfm/ft	cubic feet per minute per foot
cfm/ft^2	cubic feet per minute per square foot
cfm/ft^3	cubic feet per minute per cubic foot
cm/d	centimetres per day
cm/hr	centimetres per hour
cm/s	centimetres per second
cm/wk	centimetres per week
cu ft/mil. US gal	cubic feet per million U.S. gallons
d	days
ft	feet
ft/100 ft	feet per 100 feet
ft/ft	feet per foot
ft/h	feet per hour
ft/min	feet per minute
ft/s	feet per second
ft$^{1/2}$/s	square root of feet per second

UNITS	DEFINITION
ft^2	square feet
ft^2/cap	square feet per capita
ft^2/ft^3	square feet per cubic foot
$ft^3/(min \cdot ft^2)$	cubic feet per minute per square foot
g/(cap·d)	grams per capita per day
g/(m·s)	grams per metre per second
$g/(m^2 \cdot s)$	grams per square metre per second
g/kg	grams per kilogram
$g/m^2/d$	grams per square metre per day
g/m^3	grams per cubic metre
hp	horsepower
hp/1000 ft^3	horsepower per 1000 cubic feet
hr	hours
in	inches
in/ft	inches per foot
in/hr	inches per hour
in/s	inches per second
in/wk	inches per week
kg BOD_5/(kg MLVSS·d)	kilograms of five-day biochemical oxygen demand per kilogram of mixed liquor volatile suspended solids per day
kg O_2/kg BOD_5	kilograms of oxygen per kilogram of 5-day biochemical oxygen demand
kg O_2/kWh	kilograms of oxygen per kilowatt-hour
kg VS/m^2	kilograms of volatile solids per square metre
kg/(ha·d)	kilograms per hectare per day
$kg/(m^2 \cdot d)$	kilograms per square metre per day
$kg/(m^3 \cdot d)$	kilograms per cubic metres per day
kg/(P.E.·d)	kilograms per population equivalent per day
kg/d	kilograms per day
kJ/kg	kilojoules per kilogram

UNITS	DEFINITION
kJ/m^3	kilojoules per cubic metre
km	kilometres
km/hr	kilometres per hour
kPa	kilopascals
kW	kilowatts
$kW/1000\ m^3$	kilowatts per 1000 cubic metres
L	litres
L/(cap·d)	litres per day per capita
L/(ha·d)	litres per day per hectare
L/(ha·s)	litres per second per hectare
L/(m·s)	litres per second per metre
$L/(m^2 \cdot d)$	litres per day per square metre
$L/(m^2 \cdot min)$	litres per minute per square metre
$L/(m^2 \cdot s)$	litres per second per square metre
$L/(m^3 \cdot s)$	litres per second per cubic metre
L/(mm·d)/m	litres per millimetre of pipe diameter per day per linear metre of sewer length
L/d	litres per day
L/m^2	litres per square metre
L/m^3	litres per cubic metre
L/mm diameter/100 m/hr	litres per millimetre diameter per 100 metres per hour
L/s	litres per second
L/W ratio	length-to-width ratio
lb O_2/ lb BOD_5	pounds of oxygen per pound of 5-day biochemical oxygen demand
$lb/(1000\ ft^3 \cdot d)$	pounds per day per 1000 cubic feet
lb/(ac·d)	pounds per day per acre
$lb/(ft^2 \cdot d)$	pounds per day per square foot
$lb/(ft^2 \cdot hr)$	pounds per hour per square foot
$lb/(ft^2 \cdot min)$	pounds per minute per square foot

UNITS	DEFINITION
$lb/(ft^3 \cdot d)$	pounds per day per cubic foot
$lb/(hp \cdot hr)$	pounds per horsepower per hour
$lb/100\ ft^2$	pounds per 100 square feet
$lb/1000\ ft^3$	pounds per 1000 cubic feet
lb/1000 lb of MLVSS/hr	pounds per thousand pounds of mixed liquor volatile suspended solids per hour
lb/d	pounds per day
lb/P.E./d	pounds per population equivalent per day
m	metres
m/d	metres per day
m/hr	metres per hour
m/min	metres per minute
m/s	metres per second
$m^{1/2}/s$	square root of metres per second
m^2	square metres
m^2/cap	square metres per capita
m^2/m^3	square metres per cubic metre
m^3	cubic metres
$m^3/(ha \cdot d)$	cubic metres per hectare per day
$m^3/(m \cdot d)$	cubic metres per metre per day
$m^3/(m^2 \cdot d)$	cubic metres per square metre per day
m^3/d	cubic metres per day
$m^3/min/1000\ m^3$	cubic metres per minute per 1000 cubic metres
m^3/s	cubic metres per second
mg/g of MLVSS/hr	milligrams per gram of mixed liquor volatile suspended solids per hour
$mg/(m^2 \cdot s)$	milligrams per square metre per second
mg/L	milligrams per litre
mi	miles
MJ/dry t	megajoules per dry U.S. ton

UNITS	DEFINITION
mL	millilitres
mL/m^3	millilitres per cubic metre
mm	millimetres
mm/s	millimetres per second
mUSgd	million U.S. gallons per day
$mW{\cdot}s/cm^2$	milliwatt-seconds per square centimetre
oz/(cap·d)	ounces per capita per day
psi	pounds per square inch
US gal	U.S. gallons
US gal/(ac·d)	U.S. gallons per acre per day
US gal/(cap·d)	U.S. gallons per capita per day
US gal/(ft·s)	U.S. gallons per foot per second
US gal/(ft^2·d)	U.S. gallons per day per square foot
US gal/(ft^2·s)	U.S. gallons per second per square foot
US gal/(in·d)/ft	U.S. gallons per inch of pipe diameter per day per linear foot of sewer length
US gal/1000 ft^3	U.S. gallons per 1000 cubic feet
US gal/d	U.S. gallons per day
US gal/ft^2	U.S. gallons per square foot
US gal/hr/ft	U.S. gallons per hour per foot
US gal/lb	U.S. gallons per pound
US gal/s	U.S. gallons per second
USgpd/ft	U.S. gallons per day per foot
USgpd/ft^2	U.S. gallons per day per square foot
USgpm	U.S. gallons per minute
USgpm/ft^2	U.S. gallons per minute per square foot
W/m^3	watts per cubic metre
W	watts

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APPENDIX III

ABBREVIATIONS

ABBREVIATION	DEFINITION
ADF	Average Daily Flow
ASP	Activated Sludge Process
ATAD	Autothermal Thermophilic Aerobic Digestion
BAF	Biological Aerated Filter
BMP	Best Management Practice
BNR	Biological Nutrient Removal
BOD_5	Five-day Biochemical Oxygen Demand; equivalent to five-day Total Biochemical Oxygen Demand ($TBOD_5$)
BPR	Biological Phosphorous Removal
CAS	Conventional Activated Sludge
CCTV	Closed-Circuit Television
$CBOD_5$	Five-day Carbonaceous Biochemical Oxygen Demand
Class EA	Municipal Engineers Association's Municipal Class Environmental Assessment
COD	Chemical Oxygen Demand
C of A	Certificate of Approval
CIP	Clean-in-Place
CSO	Combined Sewer Overflow
DAF	Dissolved Air Flotation
DO	Dissolved Oxygen
DS	Dry Solids
DPHF	Design Peak Hourly Flow
D/W	Depth-to-Width Ratio
DWF	Dry Weather Flow
EBCT	Empty Bed Contact Time
ED50	Effective Dose - 50th Percentile

ABBREVIATION	DEFINITION
ESR	Environmental Study Report
FBI	Fluidized Bed Incineration
F/M	Food-to-Microorganism Ratio
F/M_v	Food-to-Microorganism Ratio Based on MLVSS
FWS	Free Water Surface
GAC	Granular Activated Carbon
GBT	Gravity Belt Thickener
GC	Gas Chromatography
GFCI	Ground Fault Circuit Interruption
GP	Grinder Pump
HDD	Horizontal Directional Drilling
HDPE	High Density Polyethylene
HMI	Human Machine Interface
HRT	Hydraulic Retention Time
HVAC	Heating, Ventilating and Air Conditioning
I/I	Infiltration and Inflow of the Sewer System
IFAS	Integrated Fixed Film Activated Sludge
IFS	Integrated Fixed-film System
LSSDS	Large Subsurface Sewage Disposal System
LPHO	Low Pressure High Output (UV Lamp)
LP/HI	Low Pressure/High Intensity (UV Lamp)
LP/LI	Low Pressure/Low Intensity (UV Lamp)
L/W	Length-to-Width Ratio
MF	Microfiltration
MLSS	Mixed Liquor Suspended Solids (in the Reactor)
MLVSS	Mixed Liquor Volatile Suspended Solids (in Reactor)
MP	Medium Pressure (UV Lamp)
MP/HI	Medium Pressure/High Intensity (UV Lamp)
MSDS	Material Safety Data Sheet

ABBREVIATION	DEFINITION
MTZ	Mass Transfer Zone
NOD	Nitrogenous Oxygen Demand
NPS	Nominal Pipe Size
ODWQS	Ontario Drinking Water Quality Standards
OPSS	Ontario Provincial Standards Specifications for Roads and Public Works
ORP	Oxidation-reduction Potential
OU	Odour Unit
OUR	Oxygen Uptake Rate
PAC	Powdered Activated Carbon
PCN	Process Control Network
PLC	Programmable Logic Controller
PE	Population Equivalent
PFD	Process Flow Diagrams
P&ID	Process and Instrumentation Diagrams
PVC	Polyvinyl Chloride
PWQO	Provincial Water Quality Objectives
RAS	Return Activated Sludge
RBC	Rotating Biological Contactors
RDT	Rotary Drum Thickener
RTB	Retention Treatment Basins
RTC	Real-Time Control
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control And Data Acquisition
SDGS	Small Diameter Gravity Sewers
SI	International System of Units
SG	Specific Gravity
SOR	Surface Overflow Rate
SRT	Solids Retention Time (or Sludge Age)
SS	Settleable Solids

ABBREVIATION	DEFINITION
SSF	Subsurface Flow
STEP	Septic Tank Effluent Pumping System
STP	Sewage Treatment Plant
SVI	Sludge Volume Index
SWD	Sidewater Depth
TAN	Total Ammonia-Nitrogen
TKN	Total Kjeldahl Nitrogen – Total of Organic Nitrogen + Total Ammonia-Nitrogen
TF/SC	Trickling Filter/Solids Contact
TOC	Total Organic Carbon
TS	Total Solids
TSS	Total Suspended Solids
TVS	Total Volatile Solids
TWL	Top Water Level
UF	Ultrafiltration
UPS	Uninterruptible Power Supply
UTM	Universal Transverse Mercator
UV	Ultraviolet
VAC	Volts Alternating Current
VE	Value Engineering
VFD	Variable Frequency Drive
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WWF	Wet Weather Flow

APPENDIX IV

ACRONYMS

ACRONYM	DEFINITION
ANSI	American National Standards Institute
APHA	American Public Health Association
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
AWWARF	American Water Works Association Research Foundation
CGA	Canadian Gas Association
CSA	Canadian Standards Association
CSCE	Canadian Society for Civil Engineering
MEA	Municipal Engineers Association (of Ontario)
MNR	Ministry of Natural Resources (of Ontario)
MOE	Ministry of the Environment (of Ontario)
NEMA	National Electrical Manufacturers Association
NWRI	National Water Research Institute
PEO	Professional Engineers Ontario
U.S. EPA	United States Environmental Protection Agency
WEF	Water Environment Federation

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APPENDIX V

SUMMARY OF MOE DESIGN CRITERIA FOR CONVENTIONAL SEWAGE TREATMENT PROCESSES

Process : *Primary Sedimentation Tanks*

Parameter	Units	Process Type/Condition		Reference
		Without WAS	With WAS Co-thickening	
Surface Overflow Rate (SOR) [at design average daily flow]	$m^3/(m^2 \cdot d)$	30 - 40	25 - 30	Table 11-1
SOR (at design peak daily flow)	$m^3/(m^2 \cdot d)$	60 - 80	50 - 60	Table 11-1

Note:

1. WAS means waste activated sludge

Process : *Aeration Tank of Activated Sludge Processes*

Parameter	Units	Process Type/Condition			Reference
		Conventional AS w/o nitrification	Conventional AS w/ nitrification	Extended Aeration	
BOD_5 Loading	$kg/(m^3 \cdot d)$	0.31 – 0.72	0.31 – 0.72	0.17 – 0.24	Table 12-1
F/M_v	d^{-1}	0.2 – 0.5	0.05 – 0.25	0.05 – 0.15	Table 12-1
HRT (min.)	hours	6	6	15	Table 12-1
Return Sludge Rate	% of Q_{avg}	25 – 100	50 – 200	50 – 200	Table 12-1
SRT (min.)	days	4 – 6	>4 at 20°C >10 at 5°C	>15	Table 12-1
Oxygen Demand	kg O_2	1.0 per kg BOD_5	1.0 per kg BOD_5 + 4.6 per kg TKN	1.5 per kg BOD_5 + 4.6 per kg TKN	Table 12-1
MLSS	mg/L	1,000 – 3,000	3,000 – 5,000	3,000 – 5,000	Table 12-1

Notes:

1. AS means activated sludge. 2. F/M_v means food-to-microorganism based on volatile MLSS (i.e., MLVSS).
3. HRT means hydraulic retention time. 4. SRT means solids retention time.
5. MLSS means mixed liquor suspended solids. 6. Q_{avg} means the average daily flow rate to the sewage treatment plant.
7. TKN means Total Kjeldahl Nitrogen.

Process : *Secondary Sedimentation Tanks*

Parameter	Units	Process Type/Condition		Reference
		Rectangular	Circular	
Dimensions:				
SWD	m	3.6 - 4.6	3.6 - 4.6	Section 13.2.1
Length:Width		> 4:1	NA	Section 13.2.1
Rapid Sludge Removal	m	NA	> 18	Section 13.3.2

Parameter	Units	CAS, Step Aeration, Complete Mix, etc.	EA, Single-Stage Nitrification	Chemicals added to MLSS for effluent TP <1.0 mg/L	Reference
Loading					
SOR (at DPHF)	$m^3/(m^2 \cdot d)$	50	40	37	Table 13-1
Peak Daily Solids Loading Rate (w/ RAS)	$kg/(m^2 \cdot d)$	240	170	240; w/o nitrification 170; w/ nitrification	Table 13-1

Note:

1. SWD means sidewater depth.
2. CAS means conventional activated sludge.
3. EA means extended aeration.
4. SOR means surface overflow rate.
5. DPHF means design peak hourly flow.
6. RAS means return activated sludge.

Process : *Effluent Disinfection*

Parameter	Units	Process Type/Condition			Reference
		Secondary Effluent	Tertiary Filtered Effluent		
Chlorine Dosage	mg/L	1 – 12	1 - 6		Table 14-1
Parameter	**Units**	**Process Type/Condition**			**Reference**
		LP/LI	**LP/HI**	**MP/HI**	
UV Dosage	mW·s/cm^2	30	30	30	Section 14.4.3
UV – effective lamp life	h	8,000 – 13,000	5,000 – 12,000	5,000 – 8,000	Sections 14.4.2.2 - 14.4.2.4

Notes:

1. LP/LI means low pressure / low intensity.
2. LP/HI means low pressure / high intensity.
3. MP/HI means medium pressure / high intensity.
4. Chlorine dosage based on a 30-minute contact time at design average daily flow.
5. Typical chlorine and UV dosages to ensure a monthly geometric mean density of less than 200 *E. coli* organisms per 100 mL.

Process : *Chemical Addition for Phosphorus Removal*

Parameter	Units	Process Type/Condition			Reference
		Alum	Ferric Chloride	Lime	
Typical Dosage Rates (for target effluent TP of 1.0 mg/L)	mg/L	110 - 225	6 - 30	40 - 400	Section 15.1.2.1

Process : *Tertiary Filtration*

Parameter	Units	Process Type/Condition			Reference
		Shallow Bed	Deep Bed		
Filtration Rates (at DPHF)	L/m²·s	2.1	3.3		Section 15.2.4
Solids Loading Rate (at DPHF)	mg/m²·s	51	83		Section 15.2.4
Parameter	**Units**	**Process Type/Condition**			**Reference**
		Min. Rate	**Min. Bed Expansion**	**Min. Duration**	
Backwash		10 L/m²·s	20%	10 minutes	Section 15.2.5

Process : *Estimated Sludge Quantities*

Parameter	Units	Process Type/Condition			Reference
		CAS w/ Primaries	CAS w/o Primaries		
Biological Sludge Production	kg TSS/kg BOD_5 removed	0.70	0.85		Section 16.1.1
Parameter	**Units**	**Process Type/Condition**			**Reference**
		CAS w/ P Removal	**EA w/ P Removal**		
			Undigested WAS	**Holding Tank Waste Sludge**	
Undigested Sludge Solids Production Concentration	g/cap·d % TS	100 4.0 [2.0 – 6.5]	55 0.9 [0.4 – 1.9]	50 2.0 [0.4 – 4.5]	Table 16-1
Digested Sludge Solids Production Concentration	g/cap·d % TS	68 4.0 [2.0 – 6.0]			Table 16-1

Notes:

1. CAS means conventional activated sludge (with upstream primary treatment).
2. EA means extended aeration.
3. Sludge concentrations expressed as average values, with ranges in parentheses.

Process : *Digestion (Mesophilic)*

Parameter	Units	Anaerobic	Aerobic	Reference
Retention Time HRT (min.) SRT (min.)	 days days	 ≥ 15 ≥ 15	 -- ≥ 45 (total)	 Section 16.2.2.3 Section 16.3.2
VS Loading	$kg/m^3 \cdot d$	0.65 (to low rate primary digester) 1.6 (to high rate primary digester)	1.6 (to first stage digester)	Section 16.2.4 Section 16.3.1
Sidewater Depth	m	≥ 6.1	3.6 – 4.6	Section 16.2.2.1 Section 16.3.1

Note:

1. HRT means hydraulic retention time.
2. SRT means solids retention time.
3. VS means volatile solids.
4. Aerobic SRT includes the activated sludge process and the aerobic digestion process SRT.

Process : *Sludge Thickening*

Process Type	Expected Performance			Reference
	Sludge Type	Sludge Concentration (%TS)	Solids Capture (%)	
Centrifugation - basket centrifuge - disc-nozzle centrifuge - solid bowl centrifuge	Waste Activated (WAS)	 8 – 10 4 – 6 5 – 8	 80 – 90 80 – 90 70 - 90	Table 17-1
Gravity Belt Thickener (GBT)	WAS	4 – 8	≥ 95	Table 17-1
Rotary Drum Thickener (RDT)	WAS	4 – 8	≥ 95	Table 17-1
Gravity Thickener	Raw Primary Raw Primary + WAS WAS Digested Sludges	8 – 10 5 – 8 2 – 3 8 – 14		Table 17-1
Dissolved Air Flotation (DAF)	WAS	4 – 6	≥ 95	Table 17-1

Process : *Sludge Dewatering*				
Process Type	**Sludge Type**	**Expected Performance**		**Reference**
		Sludge Concentration (%)	**Solids Capture (%)**	
Belt Filter Press	Undigested Primary +WAS Digested Primary + WAS WAS	14- 25 14 - 25 10 - 15	85 - 95	Table 17-2
Centrifuge (Solid Bowl)	Undigested Primary +WAS Digested Primary + WAS WAS	15 - 30 15- 30 12 - 15	95 - 99	Table 17-2
Filter Press	Undigested Primary +WAS Digested Primary + WAS WAS	30 - 50 35 - 50 25 - 50	90 - 95	Table 17-2
Vacuum Filter	Undigested Primary +WAS Digested Primary + WAS WAS	10 - 25 15 - 20 8 - 12	90 - 95	Table 17-2

APPENDIX VI

SUMMARY OF MAJOR CHANGES BETWEEN 1984 AND 2008 MOE DESIGN GUIDELINES FOR SEWAGE WORKS

The MOE Design Guidelines for Sewage Works have been updated to include new sewage collection and treatment technologies. The information was obtained from other guidelines, including: the Ten States Standards and design guidelines from other States and Provinces in North America. The following main additions have been made to the previous version of the guidelines:

- Chapter 3 General Design Considerations: sections on Technology Development and Sewage Treatment Plant Capacity, including Re-Rating.
- Chapter 4 Odour Control: including Odour Production and Control methods.
- Chapter 5 Design of Sewers: sections on New Installations/Construction Technologies (i.e., horizontal drilling, micro-tunneling, and pipe bursting), Alternative Sewer Types (i.e., pressure and vacuum sewers, and septic effluent systems), and Sewer System Rehabilitation.
- Chapter 7 Sewage Pumping Stations: including Forcemain Rehabilitation Techniques.
- Chapter 8 Design Considerations for Sewage Treatment Plants: section on Basis of Process Selection.
- Chapter 9 Instrumentation and Control: sections on Process Narratives and Basis of Control.
- Chapter 10 Preliminary Treatment: sections on Fine Screening, Microscreening and Vortex Grit Removal.
- Chapter 12 Biological Treatment: sections on Activated Sludge Selectors, Other Biological Systems (i.e., SBRs, RBCs, BAF, IFF, MBR, BNR and Intermittent Sand Filters).
- Chapter 14 Disinfection: sections on Dechlorination, UV Irradiation, and Ozonation.
- Chapter 15 Supplemental Treatment Process: sections on High Rate Effluent Filtration, Microscreening, Membranes, and Natural Systems.
- Chapter 16 Sludge Stabilization: sections on Thermophilic Anaerobic Digestion, ATAD, Alkaline Stabilization, Composting, Thermal Drying and Incineration.
- Chapter 17 Sludge Thickening and Dewatering: sections on Gravity Belt Thickeners and Rotary Drum Thickeners.

The following are new chapters added:

- Chapter 6 Challenging Conditions Affecting Servicing
- Chapter 19 Co-Treatment of Septage and Landfill Leachate at Sewage Treatment Plants
- Chapter 21 Control and Treatment of Combined Sewer Overflows
- Chapter 22 Large Sub-surface Sewage Disposal Systems

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APPENDIX VII

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