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Disinfection and supplement treatment processes

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 sitespecific 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 dechlorination 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 STP 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

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 STP 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 STP. 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 Cl2 (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 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 highintensity 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 STP. 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 buildup 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 lowpressure 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 (SBR)]. In SBR 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

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 (STP) are:

- Pre-Precipitation - the addition of chemical to the pretreated (i.e., screened and degritted) raw sewage prior to primary clarification. Preprecipitation 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 dualpoint 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. Postprecipitation 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 nonaerated 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 nonuniform 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·min) (3 USgpm/ft^2) for shallow bed single media systems and should not exceed 3.3 L/(m^2·min) (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·min) (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 corrosionresistant 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·min)[0.083 US gal/(ft^2·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 nonnitrified 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 yearround 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·min) [0.02 to 0.05 US gal/(ft^2·s)], removal efficiency is significantly decreased at loadings in excess of 1.3 L/(m^2·min) [0.03 US gal/(ft^2·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 STP 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 welltreated 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·min) (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·min) (12 to 25 USgpm/ft^2) of surface area. Additionally, supplemental surface wash at 12.8 to 25.7 L/(m^2·min) (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 onsite; 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 $CBOD_5$ 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·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·d) [0.02 lb/(ft^2·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 °C (50 °F) and should not be expected when water temperatures fall below 4 °C (39 °F). 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 °C (70 to 86 °F). An air temperature of -3 °C (27°F) for 12 hours will destroy the leaves and exposure to -5 °C (23 °F) 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 °C (45 °F).

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 infiltrationpercolation 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).
- he 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	Period Between Irrigation	Recommended Application

	Amount (cm) [in]	Applications (days)	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]

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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