STANDARDS AND GUIDELINES FOR MUNICIPAL WATERWORKS, WASTEWATER AND STORM DRAINAGE SYSTEMS

Alberta
ENVIRONMENTAL PROTECTION

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Environmental Assessment Division
Environmental Service

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December 1997

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FOREWORD

Alberta Environmental Protection (AEP) considers the establishment of standards and guidelines for municipal waterworks, wastewater and storm drainage facilities an integral part of our regulatory program directed at ensuring public health and environmental protection.

In accordance with the Potable Water Regulation (122/93) and the Wastewater and Storm Drainage Regulation and Wastewater and Storm Drainage (Ministerial) Regulation (119/93 and 120/93), a waterworks system, a wastewater system, and a storm drainage system must be designed so that they meet, as a minimum:

i) The standards and design requirements set out in the latest edition of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AEP, or

ii) Any other standards and design requirements specified by the Director

AEP last revised its standards and guidelines for municipal waterworks, wastewater and storm drainage systems in 1988. Early in 1995, the Department commenced a comprehensive stakeholder driven process to review and revise the 1988 standards and guidelines. The purpose of this publication is to provide comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable. The owners and/or their engineering consultants are responsible for the detailed project design and satisfactory construction and operation of safe and reliable systems.

These standards and guidelines supersede the March, 1988, Standards and Guidelines for Municipal Water Supply, Wastewater and Storm Drainage Facilities. The revised standards and guidelines, where they are more stringent than the March, 1988 Standards and Guidelines, are not intended to be retroactive; ***they shall be applied to new or expanded waterworks, wastewater and storm drainage systems only***. The owners may opt to upgrade their systems to meet the new standards and guidelines; otherwise the March, 1988 Standards and Guidelines will continue to be applied to existing facilities.

In developing the new standards and guidelines, AEP invited recognized waterworks and wastewater experts within the province to participate in the initial development and drafting stage. Representatives from the municipalities, engineering consultants, academia and environmental groups participated in three Advisory/Working Groups (A/WG) to guide and direct the development of the standards and guidelines for: Waterworks Systems; Wastewater Systems; and Storm Drainage Facilities. A Departmental A/WG also provided guidance and direction in the development of Operating and Monitoring Requirements and Guidelines for Waterworks and Wastewater Systems.

The general consensus with the A/WGs was that when it comes to activities and/or system components that directly relate to public health or environmental protection, Alberta Environmental Protection should outline the specific performance and design standards that have to be met, including the monitoring and reporting requirements that must be submitted. For all other activities, AEP should develop standards and/or guidelines, and the owners being free to choose the most efficient and cost effective solution.

The following is an explanation and outline of the four types of requirements for municipal waterworks, wastewater and storm drainage systems:

1. Performance Standards

Performance standards are either narrative criteria or numerical limits for a number of specific parameters which are required to meet a particular environmental quality or public health objective. These are mandatory requirements with which owners are expected to comply. Sections 2.0 and 3.0 provide the performance standards for waterworks systems and wastewater systems respectively. (Note: Facilities meeting the 1988 Standards and Guidelines are grandfathered from the performance standards unless they undergo major retrofits or expansions, in which case the new standards will be applied.)

2. Design Standards

These are minimum standards for design, construction and operation that ensure a particular environmental quality or public health objective. Alternative approaches may be used, if it can be demonstrated that there are better ways of achieving the same objectives. Where it is demonstrated that an alternative approach achieves the same objectives, the design standards will be duly revised in future editions. These are mandatory requirements with which municipalities are expected to comply. Sections 4.0 and 6.0 provide the design standards for waterworks systems and wastewater systems respectively. (Note: Facilities meeting the 1988 Standards and Guidelines are grandfathered from the design standards unless they undergo major retrofits or expansions, in which case the new standards will be applied.)

3. Design Guidelines

These are design guidelines intended to provide general guidance on how to achieve a certain level of system performance or reliability. Good engineering practices are included under this section. These are not mandatory requirements, and their use by municipalities and consultants is discretionary. Sections 5.0, 7.0, and 8.0 provide the design guidelines for waterworks systems, wastewater systems and stormwater drainage systems respectively.

4. **Operating and Monitoring Requirements and Guidelines**

These are system operations, monitoring and reporting requirements and guidelines that are essential:

i. to ensure ongoing sustained production and delivery of high quality drinking water; and
ii. to produce high quality effluents to ensure the protection of environment.

Sections 9.0 and 10.0 provide the operating monitoring requirements and guidelines for waterworks systems and wastewater systems respectively.

Performance standards included under item "1" above are generally included in the operating approval issued to the owner/operator of the system. All activities included under item "2" would require an Approval from AEP in accordance with section 2 of the Activities Designation Regulation, unless a particular activity is specifically exempted from the approval process. All activities included under item "3" would generally require a Letter of Authorization from AEP. Item "4" includes both mandatory requirements and optional guidelines.

The definitions for "owner" on pages 1-13 and 1-17 require clarification.

In accordance with the current regulations, the "owner" with respect to a waterworks, wastewater or storm drainage system for a hamlet is the local authority of the municipal district, county, improvement district or special area in which the systems are located. The new definition, as defined on pages 1-13 and 1-17, allows the option of the local authority or a co-op formed by the individual lot owners served by these systems to be the "owner". Steps have now been taken to revise the Potable Water Regulation and the Wastewater and Storm Drainage Regulation to redefine "owner" as stated on pages 1-13 and 1-17. Until the regulations are formally revised, the local authority will continue to be the "owner" of hamlet waterworks, wastewater and storm drainage systems. Proponents of new projects may check status of these changes to the regulations at the time they make the application for approval of a project that involves a hamlet waterworks, wastewater or storm drainage system.

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Waterworks Systems

Gammie, Les	-	CT Requirements, Section 4.1.5.1
	-	Laboratory Requirements, Section 6.3.7
Wobma, Paul	-	Chlorination Equipment, Section 4.1.5.2
Gratton, Norman	-	Labels and Materials Safety Data, Section 4.4.1
Kellendonk, David	-	Chemical Mixing and Coagulation, Section 6.3.1.2
	-	Flocculation, Section 6.3.1.3
	-	Sedimentation, Section 6.3.1.4(1)
Pfeifer, Brian	-	Dissolved Air Flotation, Section 6.3.1.4(2)
Wright, Ian; Tovey, Ford; Green, Taylor; and Vallance, Barry	-	Instrumentation and Controls, Section 6.3.5

Wastewater Systems

Chua, Ken	-	Estimating Wastewater Flows, Section 7.1.1
Kwan, Alex	-	Secondary Treatment, Section 7.3.1.6
	-	Tertiary Treatment, Section 7.3.1.7
Sentis, Vern	-	Aerated Lagoons System Design, Section 7.3.2.3
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DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AEP	-	Alberta Environmental Protection
AWWA	-	American Water Works Association
BDOC	-	Biodegradable Dissolved Organic Carbon
BNR	-	Biological Nutrient Removal
BPJ	-	Best Professional Judgement
BPR	-	Biological Phosphorus Removal
BPT	-	Best Practicable Technology
CBOD	-	Carbonaceous Biochemical Oxygen Demand at 5 days and 20 ^{o}C
CFID	-	Continuous Feed and Intermittent Discharge
DAF	-	Dissolved Air Flotation
DBP	-	Disinfection By-product
DCS	-	Distributed Control System
DO	-	Dissolved Oxygen
DOC	-	Dissolved Organic Carbon
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
IFID	-	Intermittent Feed and Intermittent Discharge
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWI	-	Groundwater subject to direct surface influences
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
MAC	-	Maximum Acceptable Concentration
MLSS	-	Mixed Liquor Suspended Solids
NH_3-N	-	Ammonia nitrogen
NSF	-	National Sanitation Foundation
NTU	-	Nephelometric Turbidity Unit
ORP	-	Oxidation Reduction Potential
OU	-	Odour Unit
PLC	-	Programmable Logic Controllers
QA/QC	-	Quality Assurance/Quality Control
RBC	-	Rotating Biological Contactor
SAR	-	Sodium Adsorption Ratio
SBR	-	Sequencing Batch Reactor
SRT	-	Sludge Retention Time
TBOD	-	Total Biochemical Oxygen Demand at 5 days and 20 ^{o}C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
UFRV	-	Unit Filter Run Volume
UV	-	Ultra Violet

Average daily design flow (water and wastewater) - The product of the following:

1. design population of the facility, and

2. the greatest annual average per capita daily flow which is estimated to occur during the design life of the facility.

Co-op - An organization formed by the individual lot owners served by a waterworks system, wastewater system or storm drainage system.

Granular Filter Media:

1. Effective Size (D_{10}) - Size of opening that will just pass 10% of representative sample of the granular filter media.

2. Uniformity Coefficient - A ratio of the size opening that will just pass 60% of the sample divided by the opening that will just pass 10% of the sample.

Groundwater - All water under the surface of the ground.

GWI - Any water beneath the surface of the ground, which AEP determines has the following characteristics:

Significant occurrence of insects of other microorganisms, algae, pathogens such as Giardia lamblia or viruses; or

Significant and relatively rapid shifts in water characteristics such as turbidity, temperature, conductivity, or pH closely correlating to climatological or surface water conditions.

Maximum daily design flow (water) - maximum 3 consecutive day average of past recorded flows, times the design population of the facility. If past records are not available, then 1.8 to 2.0 times the average daily design flow.

Maximum hourly design flow (water) - 2.0 to 5.0 times the maximum daily design flow depending on the design population.

Maximum monthly average daily design flow (wastewater) - The product of the following:

1. design population of the facility, and

2. the greatest monthly average per capita daily flow which is estimated to occur during the design life of the facility.

Owners - Owners of the waterworks or wastewater systems as defined in the regulations.

Peak demand design flow (water) - The maximum daily design flow plus the fire flow.

Peak wastewater design flow (wastewater) - The sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area (see Section 7.1.1).

Sodium Adsorption Ratio - A ratio of available sodium, calcium and magnesium in the soil solution which can be used to indicate whether or not the accumulation of sodium in the soil exchange complex will lead to a degradation of soil structure.

$$SAR = \frac{Na}{[\frac{Ca}{2} + \frac{Mg}{2}]^{1/2}}$$

Note: *All concentrations expressed in milliequivalents per litre*

Surface water - Water in a watercourse.

Watercourse - As defined in EPEA.

1.0 A GUIDELINE FOR THE APPLICATION AND APPROVAL OF MUNICIPAL WATERWORKS, WASTEWATER AND STORM DRAINAGE SYSTEMS UNDER THE ENVIRONMENTAL PROTECTION AND ENHANCEMENT ACT (EPEA)

1.1 Introduction

This guideline has been prepared to outline the procedures for making applications to construct or operate municipal waterworks, wastewater and storm drainage systems under the new Environmental Protection and Enhancement Act.

On September 1, 1993, Alberta's new Environmental Protection and Enhancement Act with its associated regulations came into force, replacing nine former acts including the Clean Water Act.

The procedures for obtaining approvals to construct and operate municipal waterworks, wastewater and storm drainage systems are significantly different under this new legislation. For certain project approvals, the opportunity for public input must be provided and all approvals may be appealed.

1.2 The Environmental Protection and Enhancement Act (EPEA)

1.2.1 History

In the 1990 Speech from the Throne, the Alberta Government indicated that it intended to develop comprehensive new environmental legislation for the province. A five stage consultation/development process followed to develop this new legislation. This process involved:

1. development of a Mission Statement called "Alberta's Environment - Toward the 21st Century" and solicitation of public comments on this proposed Mission Statement;

2. drafting of new environmental legislation based on existing legislation and the new Mission Statement, followed by public consultation on the proposed new legislation which was called the "Environmental Protection and Enhancement Act";

3. finalizing the Environmental Protection and Enhancement Act, which consolidated eight existing environmentally related Acts and part of another Act. This Act was passed by the government in June 1992, and was then proclaimed on April 21, 1993. The Act took effect and came into force on September 1, 1993.

4. drafting Regulations in support of the new Act and then presenting the draft Regulations at a number of public forums and meetings to give stakeholders an opportunity to provide comments; and

5. finalizing the regulations in April 1993 and submitting them to Cabinet for approval and proclamation with the Act. As indicated in iii) above, the Regulations took effect on September 1, 1993.

In general, the Act specifies what is to be regulated, the regulatory process, and the enforcement policy. The Regulations provide the details on this regulatory process and its application.

1.2.2 New Regulations Under EPEA that Apply to Municipal Works

With the proclamation of the new EPEA, the Clean Water Act, the Clean Water (General) Regulations, (Municipal Plant) Regulations, and Fluoridation Regulations were repealed. These regulations, providing the mandate to regulate "municipal plants", were replaced by new regulations that deal specifically with Potable Water (Waterworks), Municipal Wastewater and Storm Drainage.

The following section describes in more detail each of these new regulations which apply to municipal waterworks, wastewater and storm drainage systems as of September 1, 1993.

The new Regulations which are relevant to municipal water supply, wastewater treatment and/or storm drainage projects include:

1. <u>Activities Designation Regulation</u> - designates and defines those activities (projects) that require approval;

2. <u>Approvals Procedure Regulation</u> - outlines the approval procedure and minimum application requirements for various types of projects including waterworks, wastewater, and storm drainage system activities;

3. <u>Potable Water Regulation</u> - outlines the requirements for drinking water supply systems;

4. <u>Substance Release (Wastewater and Storm Drainage) Regulation</u> - outlines the requirements for municipal wastewater treatment and storm drainage systems; and

5. <u>Environmental Appeal Board Regulation</u> - outlines the procedure that an Environmental Appeal Board must follow in reviewing an appeal of the issuance or non-issuance of an approval.

The following is a brief summary of the key features of each of these regulations.

1.2.2.1 Activities Designation Regulation

The general types of activities requiring an approval are listed in the Schedule of Activities in the last section of the EPEA. The Activities Designation Regulation defines these activities in more detail, and groups activities using a "Division" designation. The five (5) designated Divisions of activities are as follows:

Division 1 - Waste Management
Division 2 - Substance Release (including air emissions, municipal wastewater and storm drainage, and industrial wastewater)
Division 3 - Conservation and Reclamation
Division 4 - Miscellaneous
Division 5 - Potable Water (Waterworks)

Divisions 2 and 5 define and list those municipal activities requiring an approval under the Substance Release and Potable Water Regulations, respectively. Table 1.1 (potable water) and Table 1.2 (municipal wastewater and storm drainage) list the definitions that are of particular relevance in terms of the types of projects that are subject to the regulations and who is considered the "owner" or "person responsible" for the project.

1.2.2.2 Potable Water Regulation

The following is an overview of some of the aspects of water supplies that are addressed in the Potable Water Regulation.

1. Definition of a Waterworks System

The Potable Water Regulation pertains to "waterworks" systems. In conjunction with the Activities Designation Regulation (refer to Sec. 1.2.2.1) and the Approvals Procedure Regulation (refer to Sec. 1.2.2.4), the Potable Water Regulation regulates the approval of "waterworks" systems and the quality of drinking water in Alberta.

In the Potable Water Regulation, a "waterworks system" means any system providing potable water to a municipality, municipal development, industrial development, privately owned development or private utility, and includes:

(i) water wells, surface water intakes or infiltration galleries that constitute the water supply;
(ii) water supply lines;
(iii) on-stream and off-stream water storage facilities;
(iv) water pumphouses;
(v) water treatment plants (including watering points);
(vi) potable water transmission mains;
(vii) potable water storage facilities;
(viii) potable water pumping facilities; and
(ix) water distribution system.

The definition of "waterworks system" and the definitions of its components replace the meaning of "municipal plant" in the previous Clean Water legislation.

2. System Design, Operation and Performance Issues Addressed in the Potable Water Regulation

In addition to providing relevant definitions, identifying those activities that require an approval, and listing those persons responsible for compliance with the regulation, the Potable Water Regulation outlines the following requirements pertaining to waterworks systems:

(i) waterworks systems performance;
(ii) waterworks systems design standards;
(iii) potable water quality;
(iv) disinfection and fluoridation;
(v) approval of chemicals used;
(vi) sampling;
(vii) returns and reports;
(viii) operator certification;
(ix) offenses for non-compliance; and
(x) transition of approvals from Clean Water to the new EPEA legislation.

3. Persons Responsible for Waterworks System

Under the EPEA, the "persons responsible for a waterworks system" have been defined to include (among others) the owners and operators.
These "owners", as persons responsible defined in the Potable Water Regulation, must make application for approvals in accordance with the Approvals Procedures Regulation (refer to sec 1.2.2.4). The owners and types of waterworks activities (system components) requiring approval are defined in Table 1.1.

1.2.2.3 Wastewater and Storm Drainage Regulation

The following is an overview of some aspects of municipal wastewater and stormwater management that are addressed by the Substance Release (Wastewater and Storm Drainage) Regulation.

1. Definition of Wastewater and Storm Drainage Systems

This regulation, in conjunction with the Activities Designation and Approvals Procedure Regulations, regulates the approval of municipal wastewater and storm drainage systems, and establishes the design and effluent quality requirements for these systems.

Under the new Wastewater and Storm Drainage Regulation,

(i) A "wastewater system" means a system for collecting, treating and disposing of wastewater, and includes:

a) sewers and pumping stations that make up a wastewater collection system;
b) sewers and pumping stations that transport untreated wastewater from a wastewater collection system to a wastewater treatment plant;
c) wastewater treatment plants;
d) facilities that provide storage for treated wastewater;
e) wastewater sludge treatment and disposal facilities;
f) sewers that transport treated wastewater from a wastewater treatment plant to the place where it is disposed of; and
g) treated wastewater outfall facilities, including the outfall structures to a watercourse or any appurtenances for disposal of treated wastewater to land or to wetlands.

(ii) A "storm drainage system" means any system for collecting, storing and disposing of storm drainage, and includes:

a) sewers and pumping stations that make up the storm drainage collection system;
b) storm drainage storage, management and treatment facilities that buffer the effects of the peak runoff or improve the quality of the storm water;
c) sewers and pumping stations that transport storm drainage to the location where it is treated or disposed of; and
d) storm drainage outfall structures.

These definitions, and the definitions of the respective system components as listed in the Wastewater and Storm Drainage Regulation and the Activities Designated Regulation, replace the meaning of "municipal plant" from the Clean Water legislation.

2. System Design, Operation and Performance Issues Addressed in the Wastewater and Storm Drainage Regulation

In addition to providing definitions, listing those activities that require an approval, and identifying the persons responsible for compliance with the regulation, the Wastewater and Storm Drainage Regulation outlines the following requirements for wastewater and storm drainage systems:

(i) wastewater and storm drainage system performance;
(ii) wastewater and storm drainage design standards;
(iii) substances prohibited from discharge to wastewater and storm drainage systems;
(iv) certified operators;
(v) sampling;
(vi) returns and reports;

(vii) offences; and
(viii) transition of approvals from Clean Water to AEPEA legislation.

3. Persons Responsible for a Wastewater or Storm Drainage System

Owners and operators of wastewater or storm drainage systems are defined in the AEPEA as "persons responsible". Owners are defined in the Wastewater and Storm Drainage Regulation to address this accountability issue and also to facilitate the application and approvals process as described in the Approvals Procedure Regulations. Owners of wastewater and storm drainage systems are defined in Table 1.2.

The types and components of wastewater or storm drainage system activities requiring approval are also defined in Table 1.2.

1.2.2.4 Approvals Procedure Regulation

This new regulation will establish a consistent approvals mechanism for all activities regulated under EPEA. The basic framework for the approvals procedure, including public notice of approval applications and provisions for notice of the Director's decisions on applications, are outlined in detail in the Regulation.

The new approvals process, including new activities or amendments and renewals to existing approvals, will incorporate public input in the form of statements of concerns filed by the public in response to specific applications for approvals. (Refer to section 1.3.4.)

1.2.2.5 Environmental Appeal Board Regulation

The issuance of approvals or the decisions not to issue approvals are appealable to an Environmental Appeal Board. The Environmental Appeal Board Regulation sets out the powers and procedures of the Appeal Board including the time period within which it must hear appeals. Directly affected and sufficiently interested parties may make presentations to the Appeal Board. It is therefore the responsibility of project proponents to try and address the concerns of these parties prior to making application for an approval.

It is important to note that an appeal in itself does not suspend the original decision that was made by the Director; the Appeal Board makes its recommendation to the Minister who makes the final decision with respect to the appeal.

1.3 The Approvals Process for Municipal Waterworks, Wastewater and Storm Drainage Systems

The Potable Water and Municipal Wastewater and Storm Drainage Regulations outline those projects (activities) for which application must be made for an approval under the EPEA. As previously outlined, these activities are further defined in the Regulations.

The following section describes the new application and approvals process under the Approvals Procedure Regulation.

1.3.1 Environmental Impact Assessments (EIA)

EPEA establishes a formal environmental impact assessment review process for certain activities. Municipal waterworks and wastewater activities have been exempted from Environmental Impact Assessment requirements under EPEA. The Minister may, however, require any project to undergo an EIA. Unless there are very unique or significant potential environmental impacts associated with a proposed municipal waterworks, wastewater or storm drainage project, it is not anticipated that EIAs will be required for any of these types of projects. The basis for exempting these municipal projects from the Environmental Impact Assessment process was that concerns associated with these types of non-discretionary projects had been, and could continue to be, dealt with through the approval process.

1.3.2 Submission of the Application

The application must be made and signed by the person primarily responsible for carrying on with the activity.

Application requirements for municipal waterworks, wastewater and storm drainage activities are outlined in the Approvals Procedure Regulations and in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems.

The detailed information requirements associated with the construction and operation of municipal waterworks, wastewater and storm drainage activities that require an approval are outlined in Tables 1.4, 1.5 and 1.6.

1.3.3 Review of the Application by the Director

Under the new EPEA and the Approvals Procedure Regulation, a "complete application" is one that is sufficient to enable the Director to commence a review. The complete application has another purpose. Unless the application is for a routine activity, the information contained therein must be sufficient to provide any directly affected public with the information they require to review the project and possibly file a concern with the Director.

The Director must review each application to ensure that it contains the information necessary to be considered "complete". The general information requirements are listed in section 3(1) of the Approvals Procedure Regulation. More detailed information requirements for a particular type of activity may be contained in the schedule attached to the Regulation; this additional information may be published in guidelines. An outline of the detailed information requirements associated with waterworks, wastewater and storm drainage approval applications are presented in Tables 1.4, 1.5, and 1.6. The Director, if satisfied that some of the standard or general requirements in section 3(1) of the Regulation do not apply to a particular activity, may waive any of these requirements.

1.3.4 The Public Review Process and the Director's Decision

A major difference in the new approvals legislation is the provision for public consultation and input at the application and approval stages of projects. Many activities that involve the construction or operation of a waterworks, wastewater, or storm drainage system will require public advertising at the application and approval stages.

In the review process, the Director may consider input from those submitting concerns, require the applicant to hold public meetings, or consider additional input from any source considered appropriate.

1. Non-Routine Projects

Once the application is complete, the applicant [unless the application is for a "routine activity" as discussed in Section 1.3.4(2)] will be required to provide notice of the application in a local daily or weekly newspaper, and make copies of the complete application available to interested parties. This provides an opportunity for affected individuals or groups to relay their concerns to the Director. (Note: a "routine activity" is one that in the Director's opinion will cause little or no adverse effect on the environment.)

Once the Director issues an approval (or refuses to approve an application), the decision must be communicated in writing to the applicant and all those who submitted a concern at the application stage.

Any person who submitted a concern at the application stage may appeal the Director's decision to issue an approval; conversely, an applicant may appeal a Director's refusal to approve a proposed activity. In either of these cases, the Appeal is handled by an Environmental Appeal Board in accordance with the new Environmental Appeal Board Regulation (see Section 1.2.2.5).

2. Routine Projects

For routine projects or projects undertaken in emergency situations, the Director may waive the requirement to provide notice of application. In such cases, however, the decision of the Director to either issue an approval or not issue an approval must still be advertised. Consequently, anyone who is directly affected by the decision may appeal to the Environmental Appeal Board in accordance with the new Environmental Appeal Board Regulation.

As a matter of policy, the Director will give preference to categorizing projects as "non-routine" as described in section 1.3.4(1) above. In most cases, it may be beneficial for both the applicant and interested public to have the opportunity for consultation at the application stage.

1.3.5 Time Required for an Approval

Table III outlines the various stages of the approvals process, including the approximate time required at each stage.

1. Routine Activities

In the absence of an appeal, the total time required to process an application for a "routine" activity (refer to sec 1.3.4) is in the range of 30 to 50 working days.

An appeal, if warranted, may take 4 to 6 months, depending on the time required to convene a panel of Board members, the time it takes for the EAB to hear all valid objections, and the time taken by the Minister to make the final decision based on the EAB report.

2. Non-Routine Activities

The approvals process, excluding an appeal, will require 70 to 100 working days. The additional 40 days is required at the application stage to accommodate public concerns resulting from advertising of the proposed activity.

The appeal process, if required, will be the same as indicated in Table 1.3 and Section 1.3.5(1) above.

The proponents of waterworks, wastewater, or storm drainage activities requiring approval under EPEA are advised to account for the aforementioned approval stages when planning the project implementation schedule. An orderly planning process, accounting for the time required for public consultation, may significantly reduce the time frames outlined above. Staff from the Department of Environmental Protection are available to meet and discuss proposed activities at the initial planning stages in order to expedite an orderly approvals process.

1.3.6 Form and Duration of the Approval

Under the new EPEA process, the approval for a waterworks, wastewater, or storm drainage activity will be in the form of a <u>single environmental approval</u>. This approval will include all terms and conditions for the construction, operation, reclamation and abandonment of the activity.

Approvals, amendments to approvals or approval renewals issued for municipal waterworks, wastewater and storm drainage activities will be in effect for a maximum duration of 10 years.

1.3.7 Transition Provision for Existing Clean Water Act Approvals

The EPEA approvals process described in this guideline took effect on September 1, 1993. Therefore, any applications for approval of activities made on or after September 1 were subject to the process.

The Clean Water Act, Clean Water (General) and Clean Water (Municipal Plants) Regulations were technically in effect until repealed when EPEA takes effect. Therefore, any applications for approvals received by the Director for the construction or operation of a "municipal plant" before September 1, 1993 were processed in accordance with the Clean Water legislation. Future amendments or renewals of these Clean Water Act approvals were and will continue to be done in accordance with the EPEA process. Projects approved for construction only (Permit to Construct) under the Clean Water Act prior to September 1, 1993 that had no operating approval (Licence to Operate) under the Clean Water Act prior to September 1, 1993, must apply for and obtain an EPEA operating approval.

Approvals for activities issued under the Clean Water Act before September 1, 1993 are deemed to be approvals under EPEA. All Clean Water approvals in effect as of September 1, 1993 must be amended or renewed under EPEA by September 1, 1998.

1.3.8 Exemptions

Some municipal activities that required Clean Water Act approvals no longer require an approval under the EPEA because they are dealt with through the regulations. Activities that do not require an EPEA approval are:

1. replacement and extension of an existing water distribution system, treated water storage facility, wastewater or storm drainage collection system including pumping stations (commonly referred to as watermain or sewer projects, often for subdivision development);

2. use of a chemical for the treatment of water that is not listed in an existing environmental approval for a waterworks system, or that does not have National Sanitation Foundation approval under NSF Standard 60;

3. application of wastewater sludge to land in cases where this activity is not provided for in an existing approval for a wastewater system; and

4. use of treated wastewater for irrigation or other acceptable reuse options in cases where such use is not provided for in an existing approval for the wastewater system.

Before proceeding with any of the above projects, however, certain information must be provided to the Director in accordance with the Potable Water and Wastewater and Storm Drainage Regulations, in order to obtain written Authorization. These information requirements are outlined in Table 1.7. The application must also comply with the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in all cases.

If the applicant intends to incorporate the activities described in 1.3.8(2), 1.3.8(3) or 1.3.8(4) into the long-term operation of a waterworks or wastewater system, they will become part of the approval for the system when the approval is next renewed or amended.

TABLE 1.1

DEFINITIONS USED IN THE POTABLE WATER REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 5)

TERM USED	DEFINITION
"Act"	The Alberta Environmental Protection and Enhancement Act (AEPEA).
"Approved analytical method"	An analytical method that is in accordance with: (i) the latest edition of Standard Methods for the Examination of Water and Wastewater published by the American Public Health Association, American Waterworks Association and the Water Environment Federation; (ii) the latest edition of the Methods Manual for Chemical Analysis of Water and Wastes, Alberta Environmental Centre; or (iii) a method approved by the Director.
"Approved laboratory"	A laboratory approved by the Director.
"Certified operator"	A person who holds a valid certificate of qualification of the appropriate class issued under section 17 of the Potable Water Regulation.
"Hamlet"	An unincorporated community that has been designated as a hamlet in accordance with the Municipal Government Act, and has a waterworks system that: (i) uses as the source of its water supply: (A) surface water; or (B) ground water that requires treatment to comply with potable water requirements under section 6 of the Potable Water Regulation; (ii) has 15 or more service connections; or (iii) has 3 or more kilometres of water distribution system.
"Industrial development"	Any development on the site of a plant referred to in section 2 of the Schedule of Activities in the Act that provides potable water where the source of its water supply is: (i) surface water; or (ii) ground water that requires treatment to comply with potable water quality requirements under section 6 of the Potable Water Regulation.
"Municipality"	The geographical area of a city, town, new town, village, summer village, municipal district, improvement district, special area, rural district or settlement area within the meaning of the Metis Settlements Act.

TABLE 1.1

DEFINITIONS USED IN THE POTABLE WATER REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 5)

TERM USED	DEFINITION
"Municipal development"	Any development that consists of two or more lots and has a common waterworks system that: (i) uses as the source of its water supply: (A) surface water; or (B) ground water that requires treatment to comply with potable water requirements under section 6 of the Potable Water Regulation; (ii) has 15 or more service connections; or (iii) has 3 or more kilometres of water distribution system but does not include a city, town, new town, village, summer village, hamlet, settlement area within the meaning of the Metis Settlements Act, regional services commission, privately owned development, industrial development, watering point, or private utility.
"Owner"	With respect to waterworks system, means: (i) the local authority of a city, town, new town, village, summer village or settlement area within the meaning of the Metis Settlements Act, in which the waterworks system is located; (ii)* for a hamlet: (A) the local authority of the municipal district, improvement district, or special area in which the hamlet's waterworks system is located; or (B) a co-op formed by the individual lot owners served by the hamlet's waterworks system unless that co-op fails or ceases to exist then the local authority; (iii) the collection of individual lot owners located in a municipal development that is served by the waterworks system; (iv) in respect of a waterworks system that serves a privately owned development, the owner of the privately owned development; (v) the regional services commission that owns a waterworks system; (vi) in the case of a waterworks system that is a private utility, the owner of the private utility; (vii) the owner of an industrial development in which the waterworks system is located; or (viii) in the case of a waterworks system that is a watering point, the local authority that owns the watering point.

TABLE 1.1

DEFINITIONS USED IN THE POTABLE WATER REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 5)

TERM USED	DEFINITION
"Person responsible for a waterworks system"	Means: (i) the owner of the waterworks system; (ii) the operator of the waterworks system; (iii) the local authority that contracts to obtain potable water from the waterworks system; (iv) the local authority that grants a franchise for the supply of potable water by the waterworks system; (v) any successor, assignee, executor or administrator, receiver, receiver-manager or trustee of a person referred to in subclause (i), (ii), (iii) or (iv); or (vi) any person who acts as the principal or agent of a person referred to in subclause (i), (ii), (iii), (iv) or (v).
"Privately owned development"	A recreational development, school, mobile home park, restaurant, motel, community hall, work camp, holiday trailer park, campsite, picnic site, information centre or other similar development, including such a development owned or operated by the Government that is: (i) on a parcel of land that is not subdivided; and (ii) served by a waterworks system that uses as the source of its water supply: (A) surface water; or (B) ground water that requires treatment to comply with potable water quality requirements under section 6 of the Potable Water Regulation; but does not include a single family dwelling, a farmstead or a development that is located on land included in a condominium plan located within a city, town, new town, village, summer village or hamlet and registered under the Land Titles Act.
"Private utility"	A waterworks system owned and operated by a person other than a local authority, municipal development, industrial development or privately owned development, but does not include a system that services only a single family dwelling or a farmstead.
"Service connection"	The potable water service line from a water distribution main to the property being serviced, but for the purposes of applying the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, means the potable water service line from a water distribution main to a building.

TABLE 1.1

DEFINITIONS USED IN THE POTABLE WATER REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 5)

TERM USED	DEFINITION
"Waterworks system"	Any system providing potable water to a municipality, municipal development, industrial development, privately-owned development, private utility or watering point and includes: (i) water wells, surface water intakes or infiltration galleries that constitute the water supply; (ii) water supply lines; (iii) on-stream and off-stream water storage facilities; (iv) water pumphouses; (v) water treatment plants; (vi) potable water transmission mains; (vii) potable water storage facilities; (viii) potable water pumping facilities; or (ix) water distribution systems.
"Water distribution system"	A system of pipes, valves, fittings and appurtenances, including associated pressure reducing stations, that is used to convey potable water in a waterworks system to the service connection for a property.
"Water treatment plant"	The physical components of the waterworks system that are used to produce potable water, including components associated with the management of any wastes generated during treatment.
"Watering point"	A waterworks system owned by a local authority that provides potable water in bulk to the public and uses as the source of its supply: (i) surface water; or (ii) ground water from more than 15 meters under the surface of the ground that requires treatment to comply with potable water quality requirements under section 6 of the Potable Water Regulation.

* Note: "owner" with respect to waterworks system for a hamlet, as defined here, is subject to ministerial approval of the proposed changes to the Potable Water Regulation.

TABLE 1.2

DEFINITIONS USED IN THE SUBSTANCE RELEASE (WASTEWATER AND STORM DRAINAGE) REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 2)

TERM USED	DEFINITION
"Act"	The Alberta Environmental Protection and Enhancement Act (AEPEA).
"Approved analytical method"	An analytical method that is in accordance with: (i) the latest edition of Standard Methods for the Examination of Water and Wastewater published by the American Public Health Association, American Waterworks Association and the Water Environment Federation; (ii) the latest edition of the Methods Manual for Chemical Analysis of Water and Wastes, Alberta Environmental Centre; or (iii) a method approved by the Director.
"Approved laboratory"	A laboratory approved by the Director.
"Certified operator"	A person who holds a valid certificate of qualification of the appropriate class issued under section 3(1) of the Wastewater and Storm Drainage (Ministerial) Regulation.
"Domestic wastewater"	The wastewater that is the composite of liquid and water-carried wastes associated with the use of water for drinking, cooking, cleaning, washing, hygiene, sanitation or other domestic purposes, together with any infiltration and inflow wastewater that is released into a wastewater collection system.
"Hamlet"	An unincorporated community that has been designated as a hamlet in accordance with the Municipal Government Act.
"Industrial Development"	Any development on the site of a plant that is served by a wastewater system that: (i) discharges wastewater off the site of the development; or (ii) is designed to generate more than 50 m^3 of wastewater per day.
"Municipality"	The geographical area of a city, town, new town, village, summer village, municipal district, improvement district, special area, rural district or settlement area within the meaning of the Metis Settlements Act.
"Municipal development"	Any development that consists of two or more lots and shares a common wastewater system or storm drainage system, but does not include a city, town, new town, village, summer village, hamlet, settlement area within the meaning of the Metis Settlements Act, regional services commission, privately owned development, industrial development, or private utility.

TABLE 1.2

DEFINITIONS USED IN THE SUBSTANCE RELEASE (WASTEWATER AND STORM DRAINAGE) REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 2)

TERM USED	DEFINITION
"Owner"	With respect to a wastewater system, means: (i) the local authority of a city, town, new town, village, summer village or settlement area within the meaning of the Metis Settlements Act, in which the wastewater system or storm drainage system is located; (ii)* for a hamlet: (A) the local authority of the municipal district, improvement district or special area in which the hamlet's wastewater system or storm drainage system is located; or (B) a co-op formed by the individual lot owners served by the hamlet's wastewater system or storm drainage system unless that co-op fails or ceases to exist then the local authority; (iii) the collection of individual lot owners located in a municipal development that is served by the wastewater system or storm drainage system; (iv) in respect of a wastewater system or storm drainage system that serves a privately owned development that is not located in a municipality referred to in (i), the owner of the privately owned development; (v) the regional services commission that owns a wastewater system or storm drainage system; (vi) in the case of a wastewater system or storm drainage system that is a private utility, the owner of the private utility; or (vii) in the case of a wastewater system that is an industrial development, the owner of the plant.
"Person responsible for a wastewater system or storm drainage system"	Means: (i) the owner of the wastewater system or storm drainage system; (ii) the operator of the wastewater system or storm drainage system; (iii) the local authority that grants a franchise for the treatment and disposal of wastewater at the wastewater system; (iv) any successor, assignee, executor or administrator, receiver, receiver-manager or trustee of a person referred to in subclause (i), (ii) or (iii); or (v) any person who acts as the principal agent of a person referred to in subclause (i), (ii), (iii) or (iv).
"Plant"	All buildings, structures, process equipment, pipelines, vessels, storage and material handling facilities, roadways and other installations, used in and for any activity listed in section 2 of the Schedule of Activities in the Act, including the land, other than undeveloped land, that is used for the purposes of the activity. Note: This is not a wastewater treatment plant or industrial wastewater treatment plant.

TABLE 1.2

DEFINITIONS USED IN THE SUBSTANCE RELEASE (WASTEWATER AND STORM DRAINAGE) REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 2)

TERM USED	DEFINITION
"Privately owned development"	A recreational development, school, mobile home park, restaurant, motel, community hall, work camp, holiday trailer park, campsite, picnic site, information centre or other similar development, including such a development owned or operated by the Government that is: (i) on a parcel of land that is not subdivided; and (ii) served by a wastewater system that: (A) discharges wastewater off the site of the development; (B) is designed to generate more than 50 m^3 of wastewater per day; but does not include a single family dwelling, a farmstead, or a development that is located on land included in a condominium plan located within a city, town, new town, village, summer village or hamlet and registered under the Land Titles Act.
"Private utility"	A wastewater system or storm drainage system owned and operated by a person other than a local authority, municipal development, industrial development, or privately owned development, but does not include a system that services only a single family dwelling or a farmstead.
"Service connection"	The sewer service line from a collection sewer to the property being serviced but, for the purposes of applying the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, means the sewer service line from a collection sewer to a building.
"Sewer"	Any system of pipes, drains, pumping works, equipment, structures and other things used for the collection, transportation or disposal of storm drainage or wastewater, but does not include any building drain, plumbing or building sewer.
"Sludge"	The accumulated wet or dry solids that are separated from wastewater during treatment, including the precipitate resulting from the chemical or biological treatment of wastewater.
"Storm drainage system"	Any system for collecting, storing and disposing of storm drainage, and includes: (i) the sewers and pumping stations that make up the storm drainage collection system; (ii) the storm drainage storage, management and treatment facilities that buffer the effects of the peak runoff or improve the quality of the storm water; (iii) the sewers and pumping stations that transport storm drainage to the location where it is treated or disposed of; and (iv) the storm drainage outfall structures.

TABLE 1.2

DEFINITIONS USED IN THE SUBSTANCE RELEASE (WASTEWATER AND STORM DRAINAGE) REGULATION AND ACTIVITIES DESIGNATION REGULATION (DIVISION 2)

TERM USED	DEFINITION
"Storm drainage"	Storm drainage, possibly including industrial runoff, resulting from precipitation in a city, town, new town, village, summer village, hamlet, settlement area within the meaning of the Metis Settlements Act, municipal development or privately owned development.
"Storm drainage collection system"	Any system of sewers, valves, fittings, pumping stations and appurtenances used to collect storm drainage, up to and including the service connection.
"Storm drainage treatment facility"	Any structure or thing used for the physical, chemical or biological treatment of storm drainage, and includes any of the storage or management facilities which buffer the effects of the peak runoff.
"Wastewater system"	A system for collecting, treating and disposing of wastewater and includes: (i) sewers and pumping stations that make up a wastewater collection system; (ii) sewers and pumping stations that transport untreated wastewater from a wastewater collection system to a wastewater treatment plant; (iii) wastewater treatment plants; (iv) facilities that provide storage for treated wastewater; (v) wastewater sludge treatment and disposal facilities; (vi) sewers that transport treated wastewater from a wastewater treatment plant to the place where it is disposed of; and (vii) treated wastewater outfall facilities, including the outfall structures to a watercourse or any appurtenances for disposal of treated wastewater to land or to wetlands.
"Wastewater"	Wastewater from a city, town, new town, village, summer village, Metis settlement, hamlet, municipal development, privately owned development or private utility that is the composite of liquid and water-carried wastes associated with the domestic use of water, together with any infiltration, inflow or non-domestic wastewater, including industrial wastewater, that may enter or be released into the wastewater collection system.
"Wastewater collection system"	A system of sewers, valves, fittings, pumping stations and appurtenances used to collect wastewater, up to and including the service connection.
"Wastewater treatment plant"	Any structure or thing used for the physical, chemical, biological or radiological treatment of wastewater, and includes wastewater storage facilities, sludge treatment, storage and disposal facilities.

* Note: "owner" with respect to wastewater system or storm drainage system for a hamlet, as defined here, is subject to ministerial approval of the proposed changes to the Wastewater and Storm Drainage Regulation.

TABLE 1.3

ESTIMATED TIME REQUIRED FOR THE APPLICATION AND APPROVAL OF MUNICIPAL WATERWORKS, WASTEWATER AND STORM DRAINAGE ACTIVITIES UNDER THE ALBERTA ENVIRONMENTAL PROTECTION AND ENHANCEMENT ACT

STAGE OF APPROVAL PROCESS	REGULATORY PROCESS	APPROXIMATE TIME REQUIRED (WORKING DAYS)	
		ROUTINE ACTIVITY	NON-ROUTINE
1. Application submission to Department	application to be logged, classified and referred to Director responsible for coordinating the process	5	5
2. Assessment of application by Director to determine if application is "complete", including any correspondence to make application complete	each application must be assessed to ensure that the information submitted is sufficient to proceed with public notification and to initiate a technical review	5	5
3. Assessment of application by Director to determine if application is "routine"	if, at the Director's discretion, the activity will have little or no adverse effect on the environment, the matter may be deemed "routine"	N/A	N/A
4. Public notification (by applicant) of complete application, and submissions by concerned persons to the Director	applicant is responsible for advertising the complete application in accordance with the regulations - concerned persons have 30 days to make a submission to the Director	Not required	40
5. Initial technical review of application by Director	once complete, each application is reviewed to ensure that Department standards and requirements are met	5-10	5-20 (concurrent with stage 4)
6. Resolution of technical deficiencies between applicant and Director	applicant and Department must work to resolve any technical deficiencies in the application to expedite the Director's decision	10-20	10-20 (a part of this time may also be concurrent with stage 4)
7. Public meeting to address concerns of affected groups or persons (if required)	Director, after receiving concerns from directly affected persons, may require the applicant to hold a public meeting	should be part of steps 1 to 6	should be part of steps 1 to 6
8. Director's decision to issue (or not to issue) approval based on technical merit and input from concerned public	on the basis of the technical content of the application and concerns received by the public, the Director decides whether or not to issue the approval (which includes a new activity or an amendment)	1-5	1-5

TABLE 1.3

ESTIMATED TIME REQUIRED FOR THE APPLICATION AND APPROVAL OF MUNICIPAL WATERWORKS, WASTEWATER AND STORM DRAINAGE ACTIVITIES UNDER THE ALBERTA ENVIRONMENTAL PROTECTION AND ENHANCEMENT ACT

STAGE OF APPROVAL PROCESS	REGULATORY PROCESS	APPROXIMATE TIME REQUIRED (WORKING DAYS)	
		ROUTINE ACTIVITY	NON-ROUTINE
9. Public notification of Director's decision	in accordance with the regulations, the Director must, within 15 days, advise the applicant and concerned public of his decision	5	5
10. Appeal by applicant (Director's refusal to issue approval) or appeal by concerned persons (Director's decision to issue approval)	following the Director's decision, the applicant or concerned public directly affected by the Director's decision may object and request an appeal - person objecting has 30 days from the last public notice of the Director's decision (Note: The applicant may appeal the Director's refusal to issue an approval or if the Director cancels an approval.)	up to 30	up to 30
11. Environmental Appeal Board (EAB)	persons forward any objections directly to the EAB who take the time necessary to convene a panel of Board members and determine the validity of the objections	unspecified	unspecified
12. Appeal Hearing	once the EAB has determined that the objections warrant an appeal, the Board has up to 60 days to call a Hearing - the Board shall provide written notice of the hearing date to the Director and all those to be heard.	up to 60 (may be extended)	up to 60 (may be extended)
13. Appeal Hearing	EAB hears all valid objections	unspecified	unspecified
14. Recommendation by EAB to Minister	once the Hearing is complete, the EAB has up to 30 days to make a report to the Minister	30	30
15. Minister's decision	Minister makes the final decision on the Appeal	unspecified	unspecified

TABLE 1.4

INFORMATION THAT MUST BE SUBMITTED WITH AN APPLICATION TO CONSTRUCT OR OPERATE A NEW WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT REQUIRES AN APPROVAL UNDER AEPEA

DESCRIPTION OF NEW ACTIVITY	INFORMATION REQUIRED FOR A COMPLETE APPLICATION
A waterworks system, including any of the following components: ! water wells or surface water intakes/galleries that make up the raw water supply; ! raw water supply mains; ! raw water storage and pumping facilities; ! water treatment plant(s) ! potable water transmission mains; ! potable water storage and pumping facilities; or ! water distribution system.	! name and address of the person primarily responsible (owner) for carrying on with the activity ! statement to justify the need for proposed activity ! proposed dates for construction commencement, construction completion and commissioning (if applicable) ! assessment of technologies associated with each of the alternatives considered for the activity, including the estimated capital costs (and operating costs if applicable) for each alternative ! proposed capacity of the various system components ! description of the physical setting (or location), including any environmentally sensitive areas that may be affected by the proposed activity ! conceptual plans of topsoil conservation for a potable water transmission main or raw water supply main that falls within the meaning of a "pipeline" as defined in section 5(h), Division 3 of the Activities Designation Regulation. Pipeline information shall be submitted as outlined in the Guide for Pipelines Pursuant to the Environmental Protection and Enhancement Act and Regulations ! types and quantities of any proposed treatment chemicals ! names and classifications of the certified operator(s) ! adequacy/suitability of the proposed water supply source to meet the long-term quantity needs and to comply with water treatment requirements ! conservation measures proposed to control excess water usage or wastage ! assessment of the proposed water treatment processes and monitoring systems that will be used to ensure compliance with each potable water quality parameter ! copy of the operating manual for the waterworks system (if applicable) ! description of the methods of handling, treating and disposing of water treatment plant wastes ! assessment of alternate water supplies available in the event of a major failure of the waterworks system ! assessment of all predictable possible equipment failures or process upsets and the measures being proposed to prevent or respond to these events

TABLE 1.4

INFORMATION THAT MUST BE SUBMITTED WITH AN APPLICATION TO CONSTRUCT OR OPERATE A NEW WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT REQUIRES AN APPROVAL UNDER AEPEA

DESCRIPTION OF NEW ACTIVITY	INFORMATION REQUIRED FOR A COMPLETE APPLICATION
A wastewater system, including any of the following components: ! wastewater collection system; ! major lift stations that make up the wastewater collection system; ! sewers and pumping stations that transport untreated wastewater from the collection system to a wastewater treatment plant; ! wastewater treatment plant(s) including wastewater stabilization ponds and wastewater sludge treatment and disposal facilities; ! facilities that provide storage for treated wastewater; ! sewers that transport treated wastewater from a plant to the location of disposal; or ! treated wastewater outfall facilities, including outfall structures to a watercourse or any appurtenances for disposal of treated wastewater to land or to wetlands.	! name and address of the person primarily responsible (owner) for carrying on with the activity ! statement to justify the need for proposed activity ! proposed dates for construction commencement, construction completion, and commissioning (if applicable) ! assessment of technologies associated with each of the alternatives considered for the activity, including the estimated capital costs (and operating costs if applicable) for each alternative ! proposed capacity of the various system components ! description of the physical setting (or location) for proposed construction, including any surrounding developments or environmentally sensitive receptors that may be affected ! conceptual plans of topsoil conservation for a wastewater sewer that falls within the meaning of a "pipeline" as defined in section 5(h), Division 3 of the Activities Designation Regulation. Pipeline information shall be submitted as outlined in the Guide for Pipelines Pursuant to the Environmental Protection and Enhancement Act and Regulations ! effect that the proposed construction will have on the quantity or quality of wastewater being handled by the wastewater system ! statement regarding the adequacy of the wastewater system to meet the long-term service needs ! type and quantities of any proposed treatment chemicals ! measures being taken to control excess flows ! summary of past and/or projected raw and treated wastewater quality ! review of the wastewater treatment processes and monitoring that will be used to ensure compliance with wastewater effluent discharge limits ! description of the proposed method for sludge treatment and disposal ! copy of the operating manual for the wastewater system ! assessment of all predictable possible equipment failures or process upsets and the measures being proposed to prevent or respond to these events ! assessment of any cumulative environmental impacts resulting from the proposed activity

TABLE 1.4

INFORMATION THAT MUST BE SUBMITTED WITH AN APPLICATION TO CONSTRUCT OR OPERATE A NEW WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT REQUIRES AN APPROVAL UNDER AEPEA

DESCRIPTION OF NEW ACTIVITY	INFORMATION REQUIRED FOR A COMPLETE APPLICATION
A storm drainage system, including any of the following components: ! the storage, management, and treatment facilities that buffer the effects of peak runoff or improve the quality of the stormwater drainage; ! storm drainage collection system; ! the sewers and pumping stations that transport storm drainage from the management/treatment facilities to the location of disposal; or ! the storm drainage outfall structures.	! name and address of the person primarily responsible (owner) for carrying on with the activity ! statement to justify the need for proposed activity ! assessment of technologies associated with each of the alternatives considered for the activity, including the estimated capital costs (and operating costs if applicable) for each alternative ! proposed capacity of the various system components ! description of the physical setting (or location) for proposed construction, including any surrounding developments or environmentally sensitive receptors that may be affected ! proposed dates for construction commencement, construction completion, and commissioning (if applicable) ! conceptual plans of topsoil conservation for a storm drainage sewer that falls within the meaning of a "pipeline" as defined in section 5(h), Division 3 of the Activities Designation Regulation. Pipeline information shall be submitted as outlined in the Guide for Pipelines Pursuant to the Environmental Protection and Enhancement Act and Regulations ! effect that the proposed construction will have on the quantity and quality of storm drainage being handled by the storm drainage system ! statement regarding the adequacy of the storm drainage system to meet the long-term peak flows associated with construction of the proposed activity ! type and quantities of any proposed treatment chemicals ! measures proposed to control peak flows ! estimate of the effect that the activity will have on existing storm drainage quality ! review of the proposed monitoring systems to ensure compliance with storm drainage quality limits (where applicable) and to measure the rate of release of storm water from the management or treatment facilities ! assessment of all predictable possible equipment failures or process upsets and the measures being proposed to prevent or respond to these events ! assessment of any cumulative environmental impacts resulting from the proposed activity

TABLE 1.5

INFORMATION THAT MUST BE SUBMITTED WITH AN APPLICATION TO AMEND AN EXISTING WATERWORKS, WASTEWATER, OR STORM DRAINAGE SYSTEM APPROVAL UNDER AEPEA

EXISTING APPROVAL TO BE AMENDED	INFORMATION REQUIRED FOR A COMPLETE APPLICATION
Construction or operation of any existing waterworks, wastewater or storm drainage system components listed in Table 1.4.	! any change in the name or address of the person primarily responsible for carrying on with the activity ! statement to justify the need for the proposed amendment ! any changes in the names or classes of certified operators as listed in the existing approval ! assessment and justification of any proposed amendments to the existing approval that may involve a change in a treatment process or technology, a new raw water supply, a change in the method of wastewater disposal or a change in the method of storm drainage management ! proposed dates for construction completion for the proposed activity

TABLE 1.6

INFORMATION THAT MUST BE SUBMITTED WITH AN APPLICATION TO RENEW AN EXISTING WATERWORKS, WASTEWATER, OR STORM DRAINAGE APPROVAL UNDER AEPEA

EXISTING APPROVAL TO BE RENEWED	INFORMATION REQUIRED FOR A COMPLETE APPLICATION
Operation of a waterworks, wastewater or storm drainage system.	! name and address of the person primarily responsible for continuing on with the activity ! copies of all environmental construction or operation approvals issued in the previous approval period by the Department for the activity approval being amended ! summary of the performance of the waterworks, wastewater, or storm drainage system covering the previous approval period, including any instances of non-compliance with the terms and conditions of the approval during that period

TABLE 1.7

INFORMATION THAT MUST BE SUBMITTED AND DIRECTOR'S AUTHORIZATION THAT MUST BE OBTAINED TO CONSTRUCT, OPERATE, OR PROCEED WITH A WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT IS EXEMPT FROM THE AEPEA APPROVAL PROCESS

ACTIVITY EXEMPT FROM AN AEPEA APPROVAL	INFORMATION REQUIRED BY THE DIRECTOR
Replacement or extension of a water distribution system or treated water storage facility that is part of a waterworks system.	! engineering drawings and specifications for the proposed replacement or extension demonstrating conformance with the current edition of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! in the case of an extension, establishing to the satisfaction of the Director that: (i) increased water demand associated with the extension can be handled satisfactorily by the existing water distribution system; and (ii) increased water flow associated with the extension can be handled satisfactorily by the system's treatment and storage facilities ! written authorization from the Director to confirm that the information submitted is sufficient to proceed with the activity
The use of a chemical for the treatment of water in a waterworks system that is not listed in an existing approval for the system, or that is not listed in the National Sanitation Foundation (NSF) Standard 60.	! letter to the Director to verify that: (i) the chemical use being proposed is done in accordance with the current edition of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! written authorization from the Director that the chemical use is acceptable
Replacement or extension of sewers or pumping stations that make up a wastewater collection system or a storm drainage collection system.	! engineering drawings and specifications for the replacement or extension demonstrating conformance with the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! information to verify that the increased wastewater or storm drainage flows associated with the replacement or extension can be handled by the existing wastewater or storm drainage systems ! written authorization from the Director confirming that the information submitted is sufficient to proceed with the activity

TABLE 1.7

INFORMATION THAT MUST BE SUBMITTED AND DIRECTOR'S AUTHORIZATION THAT MUST BE OBTAINED TO CONSTRUCT, OPERATE, OR PROCEED WITH A WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT IS EXEMPT FROM THE AEPEA APPROVAL PROCESS

ACTIVITY EXEMPT FROM AN AEPEA APPROVAL	INFORMATION REQUIRED BY THE DIRECTOR
Land application of sludge to land, where the proposed sludge application activity is not provided for in the existing wastewater system approval.	! description of the proposed project, including sufficient site soils and sludge data to establish to the Director's satisfaction that the project will meet the requirements of the latest edition of: (i) the Department's Guidelines for the Application of Municipal Wastewater Sludges to Agricultural Lands; and (ii) Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! written consent to the proposed project from: (i) all owners of land that are affected by the project and who are participating in the project; and (ii) local authorities of all municipalities in which land that is affected by the project is located ! written authorization from the Director that the proposed sludge application project complies with the Department's Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! the person responsible for the wastewater system shall within two months after completion of the sludge land application, submit a report to the Director setting out the total amount of sludge applied and the exact locations of application
Use of treated wastewater for irrigation or other environmentally acceptable re-use purpose, where the proposed use is not provided for in the existing wastewater system approval.	! description of the proposed project, including sufficient treated wastewater and soils data to establish to the Director's satisfaction that the project will meet the requirements of the latest edition of the Department's Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! written consent to the proposed project from: (i) all owners of land that are affected by the project and who are participating in the project; and (ii) local authorities of all municipalities in which land that is affected by the project is located ! written authorization from the Director that the proposed irrigation project complies with the Department's Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems ! the person responsible for the wastewater system shall within two months after completion of the irrigation project, submit a report to the Director setting out the total amount of wastewater applied and the exact locations of application

TABLE 1.7

INFORMATION THAT MUST BE SUBMITTED AND DIRECTOR'S AUTHORIZATION THAT MUST BE OBTAINED TO CONSTRUCT, OPERATE, OR PROCEED WITH A WATERWORKS, WASTEWATER OR STORM DRAINAGE SYSTEM ACTIVITY THAT IS <u>EXEMPT FROM THE AEPEA APPROVAL PROCESS</u>

ACTIVITY EXEMPT FROM AN AEPEA APPROVAL	INFORMATION REQUIRED BY THE DIRECTOR
Use of certain chemicals or substances in a wastewater system or storm drainage system that may have an adverse effect on the quality of effluent or storm drainage.	! letter to the Director to justify the need for the chemical or substance addition ! any technical information or specifications required by the Director concerning the nature of the chemical or substance and the proposed dosages ! written authorization from the Director that the chemical use is acceptable

2.0 PERFORMANCE STANDARDS - WATERWORKS SYSTEMS

2.1 Potable Water Quality

Potable water in the waterworks system shall meet the health related concentration limits (Maximum Acceptable Concentrations) for substances listed in the latest edition of the Guidelines for Canadian Drinking Water Quality published by Health Canada, with the exception noted in Section 4.3.2.

2.2 Minimum Performance Requirements for Treatment Components (surface water and groundwater subject to direct surface influences - GWI)

2.2.1 Rapid Sand Filtration

In order to obtain cyst and virus reduction credits, conventional, direct or in-line filters must meet the following turbidity or particle reductions. It is recommended that in the design of a new or expanded rapid sand filtration system, consideration shall be given for the system to meet one of the following two turbidity reduction requirements:

1. Turbidity Reduction Method

(i) Turbidity requirement to obtain filtration credit as per section 2.2.4 (1)(i)

! When source turbidity is greater than or equal to 2.5 NTU, the filtered water turbidity shall be less than 0.5 NTU in at least 95% of the measurements made each calendar month.

However, the filtered water turbidity may exceed 0.5 NTU but not exceed an upper limit of 1 NTU, in no more than 4.0% of the measurements made each calendar month. The filtered water turbidity shall not exceed 2 NTU at any time.

! When source turbidity is less than 2.5 NTU, the filters shall achieve:

- An 80% reduction in source turbidity based on an average of the daily turbidity reductions measured in a calendar month. No less than 50% reduction to be achieved each day; or

- A filtered water turbidity of less than or equal to 0.1 NTU.

(ii) Turbidity requirement to obtain filtration credit as per section 2.2.4 (1)(ii)

! When source turbidity is greater than or equal to 2.5 NTU, the filtered water turbidity shall be less than 1.0 NTU in at least 95% of the measurements made each calendar month. The filtered water turbidity shall not exceed an upper limit of 3.0 NTU at any time.

! When source turbidity is less than 2.5 NTU, the filters shall achieve:

- A 60% reduction in source turbidity based on an average of the daily turbidity reductions measured in a calendar month. No less than 50% reduction to be achieved each day; or

- A filtered water turbidity of less than or equal to 0.2 NTU.

Note: i. Sampling locations and frequencies for source and filtered water turbidity measurements shall be made in accordance with Section 9.2.3.3.

ii. AEP Regional Engineer, in conjunction with the owner of the waterworks system, will use his best professional judgement (BPJ) to determine the most suitable approach for achieving the filtration credit. Some examples of situations where BPJ could be applied are included in Appendix D.

2. Particle Counting Method

The filters should achieve the following reduction of the cyst-sized particles (greater than 2 μ m) excluding disinfection:

(i) 99.68 percent (2.5 log) reduction for systems using conventional filtration; or

(ii) 99.0 percent (2.0 log) reduction for systems using direct or in-line filtration; or

(iii) an absolute value of 50 particles/mL**.**

Note: A particle counting protocol acceptable to AEP must be used. The frequency of particle counting will be established by AEP on a site specific basis.

3. Microscopic Particulate Analysis Method

The filters should achieve the following reduction of cysts or surrogate indicators of cysts:

(i) 2.5 log reduction for systems using conventional filtration.

(ii) 2.0 log reduction for systems using direct or in-line filtration.

Note: A protocol acceptable to AEP must be used. The frequency of analysis will be established by AEP on a site specific basis.

2.2.2 Slow Sand Filtration

1. Slow sand filtration shall achieve a filtered water turbidity of less than 1.0 NTU in at least 95% of the measurements made each calendar month.

2. The filtered water turbidity shall never exceed 5.0 NTU.

Note: i) If it can be demonstrated that a higher turbidity level will not endanger the health of the consumers served by the system, AEP may allow filtered turbidity to exceed 1.0 NTU, but never 5.0 NTU.

Under this circumstance, depending on raw water quality and percentage of particles/turbidity reductions, most of the 3-log reduction of <u>Giardia</u> and 4-log reduction of viruses will have to be achieved by disinfection only.

ii) Sampling locations and frequencies for source and filtered water turbidity measurements shall be made in accordance with Section 9.2.3.3.

2.2.3 Disinfection

1. For relatively unpolluted water, i.e., raw water <u>Giardia</u> levels not greater than 1 cyst/100 L, disinfection of filtered water shall be continuous to achieve between 0.5 log and 3 log reduction of <u>Giardia</u> cysts and between 2 log and 4 log reduction of viruses, depending on the removal credit allowed for filtration [reference 2.2.4(1) and 4.1.5.1]. If it is established that the raw water is polluted, i.e., <u>Giardia</u> levels greater than 1 cyst/100 L, then greater than 3-log reduction of <u>Giardia</u> cysts is required.

2. Maximum residual disinfectant concentration, measured as free chlorine shall not exceed 4.0 mg/L, or as combined chlorine shall not exceed 3.0 mg/L, anywhere in the system.

3. All water entering the distribution system shall contain a residual disinfectant concentration, measured as free or combined chlorine, of at least 0.2 mg/L at all times the distribution system serves water to the consumer.

   If combined chlorine is used as the primary disinfectant, the chlorine should be added ahead of ammonia to have sufficient free chlorine to achieve at least 2.0 log removal of viruses.

4. Residual disinfectant concentration in the distribution system, measured as total chlorine, free chlorine, or combined chlorine shall be at least 0.05 mg/L in all of the samples taken.

Note: Sampling locations and frequencies for residual disinfectant concentrations shall be made in accordance with section 9.2.3.7.

2.2.4 Filtration and Disinfection

1. Filtration Credit

i. Enhanced Turbidity Removal

If the filtration system meets the performance and operations requirements outlined in items 2.2.1(1)(i) or 2.2.2, Giardia and viruses reduction credits are as follows:

	Giardia	Viruses
Conventional	2.5 log	2.0 log
Direct or in-line	2.0 log	1.0 log
Slow sand	2.0 log	2.0 log

AEP may allow a higher level of reduction than what is listed, if it can be demonstrated to AEP's satisfaction, in accordance with Section 4.1.5.1 (7), that the higher level of reduction can be consistently achieved.

ii. Reduced Turbidity Removal

If the filtration system meets the performance and operations requirements outlined in items 2.2.1(1)(ii), Giardia and viruses reduction credits shall be based on an in-depth evaluation of the performance of the filters by use of particle counting data 2-15 um size range, or by use of microscopic particulate analysis data, as determined by AEP.

2. Minimum Cumulative Requirements

Filtration and disinfection together shall ensure:

(a) greater than 99.9 percent (3 log) reduction of Giardia cysts; and
(b) greater than 99.99 percent (4 log) reduction of viruses.

at or before the first consumer. Plants are permitted to exceed these reductions, a maximum of one day a month.

3. System non-compliance with respect to turbidity removal

For systems not meeting the turbidity requirements outlined in sections 2.2.1(1)(i), 2.2.1(1)(ii) and 2.2.2, AEP may grant a limited or no filtration credit. Plants with no filtration credit shall provide disinfection to achieve at least 3-log reduction of Giardia cysts and 4-log reduction of viruses, or issue boil advisories to the consumers served by the plants. [Note: Filtration systems that are unable to achieve a finished water turbidity of less than 1.0 NTU in at least 95 % of the measurements made each calendar month, will be required to implement a system evaluation program, and develop an optimization or upgrading plan. System evaluation program will include procedures to determine the log reduction requirement of Giardia cysts.]

4. Filtration exemption

Where a groundwater source is determined to be under the direct influence of surface water (GWI); and where the source water quality conditions are suitable to avoid filtration as determined by AEP; and where there is adequate watershed control to avoid filtration as determined by AEP; the system treatment requirements of greater than 3 log reduction of Giardia cysts and greater than 4 log reduction of viruses may be achieved by disinfection only.

Note: Filtration and disinfection credit rating may be changed, as information become available for other cysts.

2.3 Minimum Performance Requirements for Treatment Components (Groundwater)

The minimum level of treatment for groundwater not classified as GWI should be continuous and effective disinfection such that residual disinfectant concentration in the distribution system, measured as total chlorine, free chlorine or combined chlorine shall be at least 0.05 mg/L in all of the samples taken.

AEP may reduce or eliminate the requirement for disinfection for groundwater source, however, provision must still be made in the system for disinfection of water under emergency situations. Requirements for disinfection may be reduced or eliminated under the following conditions:

1. Having a satisfactory bacteriological history at the source and within the distribution system as determined by AEP; and

2. Drawing from a protected aquifer as determined by AEP.

Note: Sampling locations and frequencies for residual disinfectant concentrations shall be made in accordance with section 9.2.3.7.

2.4 Direct and Indirect Additives

To control potential adverse human health effects from products added to drinking water directly for treatment or indirectly during storage or transmission, all substances, materials or compounds that are added to or may come in contact with potable water shall conform to American National Standards Institute and National Sanitation Foundation ANSI/NSF Standards 60/61 for health effects and the manufacturer of the product shall be included in the list of approved manufacturers published by ANSI/NSF, or the product shall be approved by AEP for use in Alberta.

2.5 Fluoridation

When fluoridation is practised, adequate controls shall be maintained at all times to provide a fluoride ion concentration in treated water to meet the optimum concentration in the latest edition of GCDWQ (0.8 mg/L in the proposed 7th edition). A monthly average and daily variation shall be within ± 0.1 mg/L and ± 0.2 mg/L respectively.

3.0 PERFORMANCE STANDARDS - WASTEWATER SYSTEMS

3.1 Treated Effluent Disposal to Surface Waters

3.1.1 Treated Effluent Quality

The treated effluent quality for a wastewater treatment facility shall be based on the more stringent of the quality resulting from the "Best Practicable Technology" (section 3.1.2) or the quality required based on receiving water assessments (section 3.1.3).

Exceptions to this rule are:

- the seasonal discharges to a receiving watercourse from wastewater lagoons designed and operated in accordance with AEP standards (section 5.2.1). No receiving water assessments are required for such releases; and

- when a water quality based limit is not technically attainable. In this case, an advanced technology limit may be adopted as an interim effluent limit.

3.1.2 Best Practicable Technology Standards

Only those technologies identified in Tables 3.1 and 3.2 are considered 'Best Practicable Technologies', and the corresponding effluent standards as 'Best Practicable Technology Standards'.

3.1.3 Receiving Water Quality Based Standards

Receiving water quality based standards shall be derived by calculating the maximum amount of substances that can be discharged under worst case conditions while still maintaining instream water quality guidelines.

Detailed procedure for determining the receiving water quality based standards is included in the "Water Quality Based Effluent Limits Procedures Manual" published by AEP.

3.1.4 Disposal Criteria

Continuous discharge of effluent from treatment plants to a receiving watercourse shall be permitted if the recorded minimum mean monthly watercourse flow is ten times the total average daily discharge of treated effluent, and receiving water assessment indicates that there are no appreciable water quality impacts. However, if it can be demonstrated with a high level of certainty that no appreciable water quality impacts are projected to occur at 10:1 dilution, then discharge may be permitted at less than 10:1 dilution. Alternative methods of disposal and/or effluent storage facilities may be required if these conditions cannot be met.

Seasonal discharge of effluent from treatment plants, other than wastewater lagoons, to a receiving watercourse shall be reviewed on a site specific basis; duration and timing of discharges will be determined based on receiving water assessment.

Continuous or seasonal discharges of effluent to lakes or other stagnant water bodies are generally discouraged. Such releases shall be reviewed on a site specific basis, and will be permitted only if there are no water quality impacts. Water quality impacts will be assessed based on the anti-degradation policy, "Municipal Effluent Limits - Policy and Overview" in the Municipal Policies and Procedures Manual.

TABLE 3.1

BEST PRACTICABLE TECHNOLOGY STANDARDS FOR MUNICIPALITIES WITH CURRENT POPULATION <20,000

Type	Parameter	Standard	Sample	Comments
Secondary (mechanical)	CBOD TSS	25 mg/L 25 mg/L	composite composite	Monthly average of daily samples Monthly average of daily samples
Aerated lagoons	CBOD	25 mg/L	grab	Monthly average of weekly samples
Wastewater lagoons 2 or 4 anaerobic cells (2 day retention time in each cell) 1 facultative cell (2 month retention time) 1 storage cell (12 month retention time)	None defined	None defined	None defined	Lagoons built to the specified design configuration and drained once a year between late spring and fall do not have a specified effluent quality standard. Early spring discharges may be allowed under exceptional circumstances to comply with any local conditions. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.

Note:
1. Current population for municipalities served by the system shall be determined by taking into consideration the equivalent population for industrial waste discharges into the system. If site specific information is not available, then equivalent population for industrial wastes shall be based on 70 g CBOD per person per day.
2. Sampling frequencies are based on continuous discharge of effluent to a body of water.
3. See Table 5.2 for the basis for selecting either 0 or 2 or 4 anaerobic cells in wastewater lagoons.

TABLE 3.2

BEST PRACTICABLE TECHNOLOGY STANDARDS FOR MUNICIPALITIES WITH CURRENT POPULATION >20,000

Type	Parameter	Standard	Sample	Comments
Tertiary (mechanical)	CBOD	20 mg/L	composite	Monthly average of daily samples
	TSS	20 mg/L	composite	Monthly average of daily samples
	TP	1 mg/L	composite	Monthly average of daily samples
	NH_3-N	-	composite	Need assessed on a site specific basis
	Total Coliform	1000/100 mL	grab	Geometric mean of daily samples in a calendar month
	Fecal Coliform	200/100 mL	grab	Geometric mean of daily samples in a calendar month
Aerated lagoons	CBOD	20 mg/L	grab	Monthly average of weekly samples
	TP	1 mg/L	grab	Monthly average of weekly samples
	NH_3-N	-	grab	Need assessed on a site specific basis
	Total Coliform	1000/100 mL	grab	Geometric mean of weekly samples in a calendar month
	Fecal Coliform	200/100 mL	grab	Geometric mean of weekly samples in a calendar month
Wastewater lagoons 2 or 4 anaerobic cells (2 day retention time in each cell) 1 facultative cell (2 month retention time) 1 storage cell (12 month retention time)	None defined	None defined	None defined	Lagoons built to the specified design configuration and drained once a year between late spring and fall do not have a specified effluent quality standard. Early spring discharges may be allowed under exceptional circumstances to comply with any local conditions. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.

Note: See next page

Note:

1. Current population for municipalities served by the system shall be determined by taking into consideration the equivalent population for industrial waste discharges into the system. If site specific information is not available, then equivalent population for industrial wastes shall be based on 70 g CBOD per person per day.

2. Sampling frequencies are based on continuous discharge of effluent to a body of water.

3. Facilities producing effluent with nitrogenous oxygen demand may be required to monitor for TBOD. The need for TBOD monitoring, and subsequent limit on NH_3-N will be assessed on a site specific basis.

4. Bacteriological quality standards for total coliforms may be relaxed, if the owner demonstrates with some certainty that the wastewater being considered for disinfection is not typical of other municipal wastewaters.

5. Any sample yielding more than 400 total coliforms/100 mL shall be further investigated. Minimum action should consist of immediate re-sampling of the site.

6. Frequency of sampling for total and fecal coliforms may be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the possibility of variance is minimal.

7. See Table 5.2 for the basis for selecting either 0 or 2 or 4 anaerobic cells in wastewater lagoons.

3.2 Treated Effluent Disposal to Land

3.2.1 Wastewater Irrigation

3.2.1.1 Minimum Treatment Requirement

If wastewater irrigation is chosen as the only method for the disposal of treated effluent, the minimum wastewater treatment shall be as follows:

1. Primary treatment (anaerobic cells in series or a facultative cell) followed by at least seven month storage; or

2. Secondary treatment with or without storage

3.2.2 Treated Effluent Quality Standards

The treated effluent quality for wastewater irrigation shall meet the standards specified in Table 3.3

TABLE 3.3

TREATED EFFLUENT QUALITY STANDARDS FOR WASTEWATER IRRIGATION

Parameter	Standard	Type of Sample	Comments
Total Coliform*	<1000/100 mL	Grab	Geometric mean of weekly samples (if storage is provided as part of the treatment) or daily samples (if storage is not provided), in a calendar month
Fecal Coliform*	<200/100 mL	Grab	Geometric mean of weekly samples (if storage is provided as part of the treatment) or daily samples (if storage is not provided), in a calendar month
CBOD	<100 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
COD	<150 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
TSS	<100 mg/L	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
EC	<2.5 ds/m	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
SAR	<9	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event
pH	6.5 to 9.5	Grab/composite**	Samples collected twice annually prior to and on completion of a major application event

* For golf courses and parks only.
** Grab sample would suffice if storage is provided; Composite sample is required if storage is not provided.

3.2.2 Rapid Infiltration

3.2.2.1 Minimum Treatment Requirement

For rapid infiltration, a minimum of primary treatment shall be provided.

The system shall be designed in accordance with the joint Alberta Environmental Protection - City of Red Deer publication entitled Rapid Infiltration - A Design Manual.

3.2.3 Wetlands Disposal

3.2.3.1 Minimum Treatment Requirement

For wetlands disposal, a minimum of secondary or tertiary treatment shall be provided and the effluent quality shall meet the standards specified in tables 3.1 and 3.2.

Wetlands shall be evaluated and designed in accordance with Alberta Environmental Protection publication entitled Guidelines for the Approval and Design of Natural and Constructed Treatment Wetlands for Water Quality Improvement.

4.0 DESIGN STANDARDS - WATERWORKS SYSTEM

4.1 Treatment of Surface Water and GWI

4.1.1 Determination of GWI Sources

1. AEP shall notify the owner when a source has been identified as a potential GWI source. Until AEP has made a source determination, the municipality shall monitor in accordance with the requirements for groundwater sources outlined in Section 9.0 or as directed by AEP.

2. The owner using a source identified as a potential GWI shall provide to AEP all information necessary to determine whether the source is under direct surface water influence. Information shall include, but not be limited to:

   (i) Site-specific source water quality data;
   (ii) Documentation of source construction characteristics;
   (iii) Documentation of hydrogeology;
   (iv) Distance to surface water; and
   (v) Water quality results from nearby surface water(s) if requested by AEP.

3. Based on information provided by the owner, and any other information available, AEP shall determine which groundwater sources are subject to direct surface influences and notify the owner of the source determination.

4. The owner may modify an AEP determined GWI source to eliminate direct surface influence. In such cases, the owner shall, at a minimum:

   (i) Submit a proposed schedule for source modification to AEP for review and approval;

   (ii) Provide disinfection in accordance with section 2.2.4(3) to protect the health of consumers served by the water system until:

      (a) Modification is complete; and

      (b) AEP determines the source is no longer subject to direct surface influence.

   (iii) Comply with subsection (2) of this section upon completion of source modifications to be considered for source reclassification.

5. AEP may re-evaluate a groundwater source for direct surface influence, if conditions impacting source classification have changed, or as new information becomes available.

4.1.2 Minimum Treatment Requirements

Subject to Section 2.2.4 (3), the minimum treatment for surface water and GWI supplies shall be chemically assisted rapid sand filtration (Section 4.1.3) and disinfection, or slow sand filtration (Section 4.1.4) and disinfection.

Alternative treatment designs followed by disinfection may be acceptable to AEP only in conjunction with the policy "Unproven or Innovative/Alternative Technologies", in the Municipal Policies and Procedures Manual.

4.1.3 Rapid Sand Filtration

Chemically assisted rapid sand filtration of water is a multi-step treatment process that includes chemical mixing, coagulation, flocculation, solids separation and filtration (conventional treatment); or chemical mixing, coagulation, flocculation and filtration (direct filtration); or chemical mixing, coagulation and filtration (in-line filtration), as illustrated in Figure 1.

The system shall be designed to produce water that meets the drinking water quality standards, and the minimum performance requirements for rapid sand filtration outlined in Section 2.0 - Performance Standards - Waterworks System.

1. Chemical Mixing and Coagulation

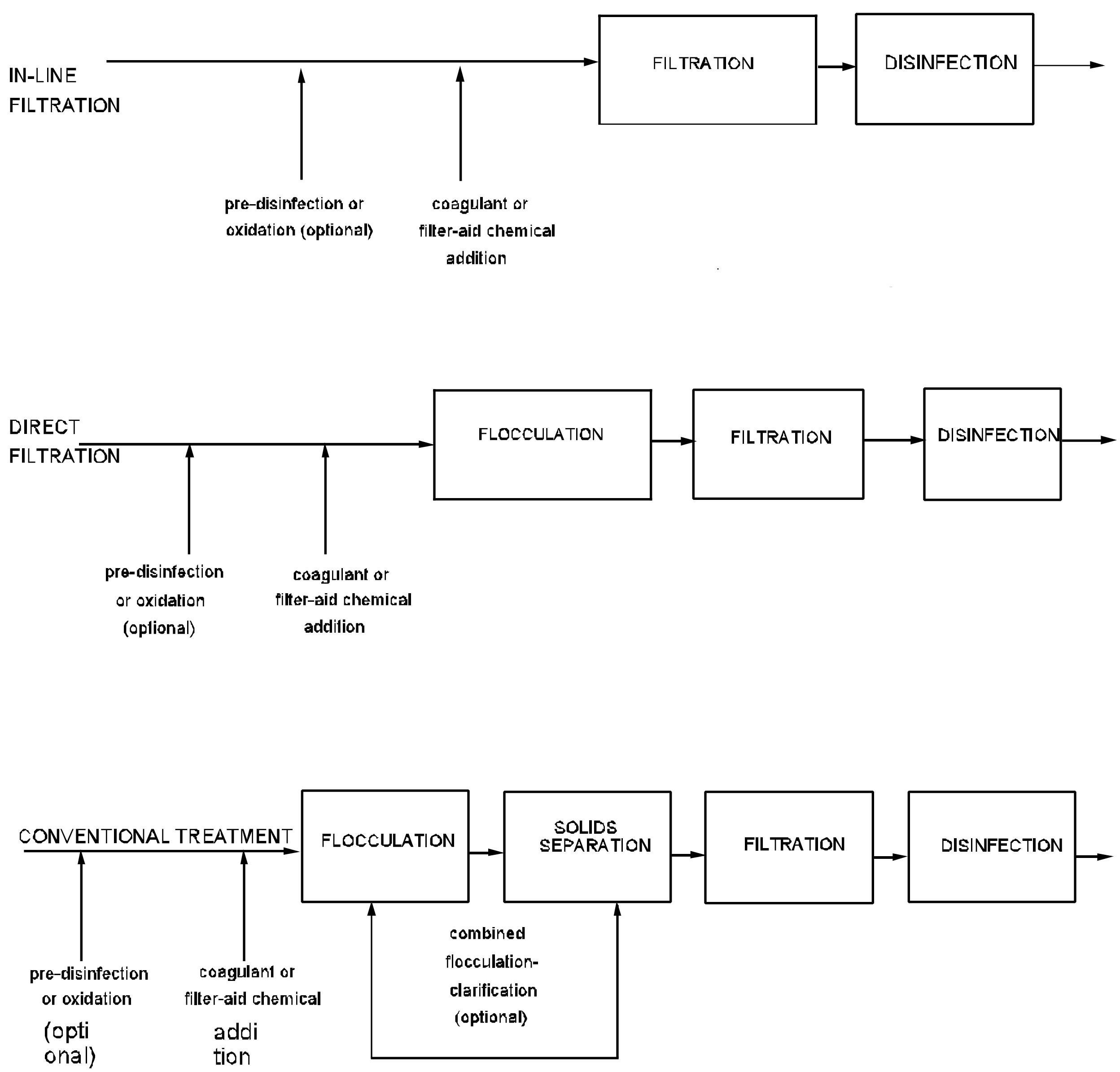
Proper design of the chemical mixing and coagulation processes is critical for effective overall performance of the treatment plant; principles discussed in this section shall be used in the design of the same. Most natural particles in water have negative surface charges; coagulation is achieved by the addition of positively charged metal ions (aluminum or iron) or cationic polymers. Chemical mixing, frequently referred to as flash or rapid-mixing is the physical process of blending or dispersing a chemical additive into an unblended stream, and is crucial to satisfactory coagulation performance. Coagulation is the process of destabilization of the charge on the suspended particles and colloids. The purpose of destabilization is to neutralize the repelling charges of the particles and allow them to become attached to other particles so that they may be removed in subsequent processes.

Coagulation by inorganic salts occurs either by "charge neutralization" or by "sweep coagulation".

Charge neutralization reactions are extremely fast, in the order of fractions of a second. Dispersion of chemical into the water as quickly as possible is of paramount importance, and high intensity mixing shall be provided. This mode of coagulation at low dosages of chemicals produces small destabilized pinpoint floc and is ideal for treating low turbidity, low alkalinity waters using direct or in-line filtration, and high turbidity, low alkalinity waters using conventional filtration.

FIGURE 4.1

TYPICAL WATER TREATMENT PROCESSES



For sweep coagulation, the coagulant is added to water in concentrations sufficiently high to cause precipitation of a metal hydroxide; and particulate removal occurs by enmeshment in the precipitate. Hydroxide formation occurs in the 1 to 7 second range and thus the short dispersion times and high intensities of mixing are not as crucial for sweep coagulation as for charge neutralization. Sweep coagulation is suitable for treating low or high turbidity, high alkalinity waters using conventional filtration.

Coagulation by organic polymers occurs by charge neutralization. However, since the competing reactions for charge neutralization and sweep coagulation do not occur simultaneously, high intensities of mixing may not be imperative when organic polymer is used as the primary coagulant. When organic polymers are used as primary coagulants they are added at the rapid mix unit and when used as a flocculant-aid they are added after the addition of the inorganic coagulant, prior to or at the flocculation unit. An optimum time of separation may be derived from jar or pilot tests. It should also be noted that at very low turbidities, polymers are ineffective coagulants.

Mixing and coagulation shall be achieved either in a separate process tank or with an in-line mixing device. The type of coagulation process shall be determined at the design stage, based on the water chemistry. Pilot studies and/or related experience shall be used to determine the energy gradients required to achieve optimum mixing.

2. Flocculation

Flocculation is a process of gentle stirring and mixing to enhance contact of destabilized particles and to build floc particles of optimum size, density and strength to be subsequently removed by settling or filtration. Polymeric flocculant aids usually anionic or nonionic, may be added to improve floc size and settling rates.

The velocity gradient, reflecting the amount of energy input to the water, is the basic design parameter for flocculation. Once mixing is achieved, the coagulated water shall be subjected to a decreasing level of energy to maximize formation of the flocs; if high turbulence or shear is applied, the formed flocs may be fragmented.

To prevent short circuiting and to permit defined zones of reduced energy input, the flocculation process shall have two or more stages/compartments unless specifically exempted by AEP. A minimum of two flocculation trains shall also be provided unless specifically exempted by AEP.

3. Solids Separation

Solids separation shall be achieved either through sedimentation or through flotation. Sedimentation is a gravity separation technique in which particles that are heavier than water settle out of a suspension. Floatation is a separation process whereby solids are transferred to the surface of a liquid through attachment to air bubbles.

(i) Sedimentation

Sedimentation may be horizontal-flow type, or upflow type, or upflow solids-contact type; the upflow solids-contact type combines chemical mixing, flocculation and up-flow sedimentation in a single unit.

Sedimentation is influenced by flocculation within the basin, and as a result, both depth and detention times, along with surface area and overflow rates become critical in the design of a sedimentation tank. To determine the overall removal of solids for a given overflow rate in practice, AEP may request an experimental analysis using a settling column be performed by the owner.

High-rate clarification with overflow rates as high as three to six times those used in conventional settling design may be allowed in conjunction with tube settlers or parallel plate settlers. The high overflow rates may be determined by undertaking a pilot study, using the proposed source water.

At least two sedimentation basins shall be provided unless specifically exempted by AEP.

(ii) Flotation

Dissolved air flotation may be used for treating waters containing large quantities of algae, and waters with low turbidity and high colours, which are typically difficult to treat by conventional sedimentation processes.

The efficiency of flotation is greatly enhanced by particle destabilization with coagulants; thus the process will be allowed only in conjunction with chemical pre-treatment and only after substantiation of the results by a pilot study.

At least two flotation basins shall be provided unless specifically exempted by AEP.

4. Filtration

Filtration represents the final particulate removal step in water treatment. Rapid sand filtration will be allowed only in conjunction with chemical pre-treatment of water.

Filters shall be gravity feed type. Pressure filters, which are more suited to operating at higher headloss, may be used under exceptional circumstances. Where AEP is not satisfied that the proposed method of filtration is capable of practically achieving the performance standards outlined in Section 2.0, the proponent will be required to substantiate the proposal by a pilot study using the source water on site.

Any rate increase or decrease on a dirty filter will cause some detriment to filtered water quality for a brief period; for this reason the filter shall be designed for continuous operation. For large facilities with a bank of filters, influent flow-controlled declining-rate filters are preferable to effluent flow-controlled constant-rate filters.

Each filter shall be equipped with an on-line turbidimeter with a recorder for continuous monitoring of effluent quality, to substantiate the efficient removal of turbidity and to permit optimization of coagulant dosages.

(i) Filtration Rate

Filter loading rate shall be considered a key factor in the design of a filtration system from both the filtrate quality and filter-run points of view. The filtration rate is dependent on the quality of raw water, the extent of pre-treatment and the characteristics and depth of the filter media. High-rate filtration will be acceptable only if it can be substantiated by undertaking a filter column study using the proposed source water.

(ii) Filter to Waste

The filter ripening period, which is the initial portion immediately after a backwash of a filter's operating cycle, typically gives rise to poorer effluent quality than later in the cycle. Filters normally will not meet the minimum performance requirement during the ripening period and the water produced during this period shall be wasted or recycled to the pre-treatment works. Water treatment plants shall be designed with this filter-to-waste provision; piping shall be designed for the capacity of that filter. Precautions should be made to prevent backflow from the filter-to-waste stream to any component of the potable water supply system.

Duration of the filter-to-waste cycle may be tied to the actual turbidity of the wasted water or to a pre-determined time for the turbidity of the wasted water to reach the required limit.

(iii) Number of Filters

The number of filter units may vary with the plant capacity. For plants with capacity greater than 150 m^3/d, a minimum of two filters shall be provided, each capable of independent operation and backwash. Where possible, three filters or more shall be provided; except during repair/emergency, all filters shall be in operation. Where only two filters are provided, each filter should have a hydraulic capacity not less than 150% of design filtration rating.

A single filter is acceptable for plants with design capacity less than 150 m^3/d.

4.1.3.1 In-line Filtration

In-line filtration is a treatment process that includes coagulant addition, rapid mixing and filtration, with flocculation occurring in the filter itself. In-line filtration may be used for source water that is consistently very low in turbidity, colour and dissolved organic carbon, and also for charge neutralization of the colloidal particles. Low chemical dosage required for charge neutralization and the resultant destabilized pin-point flocs are ideal for in-line filtration.

In-line filtration will be acceptable, only if it can be substantiated by undertaking a pilot study using the proposed source water.

4.1.3.2 Direct Filtration

Direct filtration is a treatment process that includes coagulant addition, rapid mixing, flocculation and filtration. As for in-line filtration, the process may be used for source water that is consistently low in turbidity, colour and dissolved organic carbon. Water must be suitable for charge neutralization with low coagulant dosages with the goal of forming a pin-point sized floc that is filterable, rather than a settleable floc.

Direct filtration will be acceptable only if it can be substantiated by undertaking a pilot study using the proposed source water.

4.1.3.3 Conventional Filtration

Conventional filtration treatment process includes coagulant addition, rapid mixing, flocculation, solids separation (sedimentation or floation) and filtration. Combined clarification that includes flocculation and solids separation in one unit, such as solids contact upflow clarifier, may be an alternative to flocculation-solids separation, if it can be substantiated through a pilot study.

As a rule, conventional filtration is required for all surface waters and GWI unless otherwise demonstrated through a pilot study that safe drinking water quality objectives can be met through in-line filtration or direct filtration.

4.1.4 Slow Sand Filtration

The slow sand filter is a sand filter operated at very low filtration rates without the use of coagulation in pre-treatment. The sand used is smaller than that for a rapid filter, and this, plus the low filtration rate, results in the solids being removed almost entirely in a thin layer on the top of the sand bed. This layer, composed of dirt and micro and macroorganisms from the water (i.e. the schmutzdecke) becomes the dominant filter medium as the filter cycle progresses.

When the head loss becomes excessive, the filter shall be cleaned by draining it below the sand surface and physically removing the schmutzdecke. Typical cycle lengths may vary from **3** to 6 months depending on the source water quality and the filtration rate.

The system shall be designed to produce water that meets the drinking water quality standards and minimum performance requirements for slow sand filtration outlined in Section 2.0 - Performance Standards - Waterworks System.

4.1.4.1 Requisite Conditions

Source water quality and community size are the key factors that should be considered in the selection of slow sand filtration to treat surface water.

1. Water Quality

Raw water quality determines the length of time between scraping options, and slow sand filtration shall be limited to raw waters that will permit filter runs of at least three months before terminal headloss is reached.

The mix of water quality characteristics causing headloss is unique to each situation and cycle lengths cannot be predicted without pilot plant testing. Unless specifically exempt by AEP, suitability of slow sand filtration for treatment and run-length shall be ascertained by pilot testing over the annual cycle.

Some waters with high algae content form a mat on the filter surface causing a rapid increase in headloss. Further, the decay of algae on the schmutzdecke may cause taste and odour problems in the effluent water. Thus source waters subject to algae blooms may not be acceptable for slow sand filtration unless the bloom season is short enough that only minimal interruption of normal operation is likely.

Though low turbidity of source water is not always indicative of long filter run-lengths, water with high turbidities can blind-off the filter with a layer of sediments on top of the filter bed. Thus source waters with high turbidities may not be acceptable for slow sand filtration, unless the turbidities are attenuated by the use of sedimentation basins or roughing filters.

Slow sand filtration is not effective in removing colour, and source waters with high colour may not be acceptable for slow sand filtration.

Slow sand filtration is ineffective in removing dissolved organic carbon, and because of potential disinfectant by-products, source waters with high dissolved organic carbon (DOC) may not be acceptable for slow sand filtration if chlorination is practised for disinfection. However, in some cases, ozone ahead of slow sand filtration has proven to increase run length and increase biodegradation of organic compounds, i.e. DOC converted to BDOC. This is considered an alternative technology and will be approved only in conjunction with the policy, "Unproven or Innovative/Alternative Technologies" in the <u>Municipal Policies and Procedures Manual</u>.

2. Community Size

Slow sand filtration is considered appropriate for use by "small" communities. At some point in population size, slow sand filtration becomes more expensive than rapid sand filtration. Also, at some point as population increases, communities will have the requisite resources to effectively and reliably operate a rapid sand filtration system. The point of crossover on both curves depends upon the community context, and should be considered in ascertaining the suitability of slow sand filtration for that community.

4.1.4.2 Filtration

Fluctuating load on the filter upsets the schmutzdecke, resulting in poor quality of filtered water for brief periods. For this reason the filters should preferably be designed for continuous operation.

Unlike for rapid gravity filters, continuous monitoring of effluent turbidity may not be required for slow sand filters. Thus, each slow sand filter may or may not be equipped with an on-line turbidimeter, as determined by AEP.

1. Filtration Rates

Filter loading rate is a key factor in the design of slow sand filtration system from both the filtrate quality and filter run lengths points of view. The percent removal of turbidity generally declines with increasing loading rate. Unless specifically exempt by AEP, the optimum filtration rates shall be ascertained by pilot testing over the annual cycle.

2. Filter to Waste

During the filter ripening period, following the start-up of a new filter or a re-built filter bed, filtered water will be of very poor quality. The ripening period may range from about one week to several months. The filters will not meet the minimum performance requirements during the ripening period and the water produced during this period shall be wasted or recycled to the pre-treatment works. Water treatment plants shall be designed with this filter-to-waste provision. Precautions should be made to prevent backflow from the filter-to-waste stream to any component of the potable water supply system.

During the filter-to-waste mode, the filter may be operated at high hydraulic loading rates and wasted water turbidity shall be measured daily until it reaches the acceptable level.

3. Number of Filters

Slow sand filters shall have a minimum of two or more cells so that when one is out of service for scraping or other reasons, another filter bed can continue producing sufficient amounts of water for the community.

Because of the prolonged filter ripening period, systems with two cells shall have each cell capable of producing maximum daily design flow for the community. Systems with more than two cells shall have all cells but one capable of producing maximum daily design flow.

4.1.5 Disinfection

Performance requirements for treatment with respect to disinfection of surface water are stipulated in Section 2.0 - Performance Standards - Waterworks System. This section outlines the procedures to achieve this objective and includes the design standards for the facility.

4.1.5.1 Residual Concentration/Contact Time (CT) Requirements

Disinfection to eliminate fecal and coliform bacteria may not be sufficient to adequately reduce pathogens such as Giardia or viruses to desired levels. Use of the "CT" disinfection concept is recommended to demonstrate satisfactory treatment, since monitoring for very low levels of pathogens in treated water is analytically very difficult.

The CT concept, as developed by the United States Environmental Protection Agency (Federal Register, 40 CFR, Parts 141 and 142, June 29, 1989), uses the combination of disinfectant residual concentration (mg/L) and the effective disinfection contact time (in minutes) to measure effective pathogen reduction. The residual is measured at the end of the process, and the contact time used is the T_{10} of the process unit (time for 10% of the water to pass).

CT = Concentration (mg/L) x Time (minutes)

The effective reduction in pathogens can be calculated by reference to standard tables of required CTs (see Appendices A and B).

1. Required Giardia/Virus Reduction

As referenced under Section 2.0 - Performance Standards, Waterworks System, all surface water treatment systems shall ensure a minimum reduction in pathogen levels:

- 3-log reduction in Giardia; and
- 4-log reduction in viruses.

These requirements are based on unpolluted raw water sources with Giardia levels of = 1 cyst/100 L, and a finished water goal of 1 cyst/100,000 L (equivalent to 1 in 10,000 risk of infection per person per year). Higher raw water contamination levels may require greater removals as shown on Table 4.1, however, the actual level of Giardia reduction will be determined jointly by AEP and the municipality, based on the extent and accuracy of the raw water Giardia data and the treatment efficiency of the plant. (Municipalities should endeavour to monitor raw water Giardia levels in accordance with section 9.2.2, Table 9.3, to ascertain removal goals).

TABLE 4.1

LEVEL OF GIARDIA REDUCTION

Raw Water Giardia Levels*	Recommended Giardia Log Reduction
< 1 cyst/100 L	3-log
1 cyst/100 L - 10 cysts/100 L	3-log - 4-log
10 cysts/100 L - 100 cysts/100 L	4-log - 5-log
> 100 cysts/100 L	> 5-log

*Use geometric means of data to determine raw water Giardia levels for compliance.

2. CT Credits

As referenced under Section 2.0 - Performance Standards, Waterworks System, the required log reductions are achieved by removals/inactivation due to conventional, direct or in-line filtration plus the removals/inactivation due to disinfection. Credits are provided for physical treatment for Giardia and virus reduction as shown on table 4.2 (assuming minimum performance requirements for filtration are met as per Sections 2.2.1(1)(i) and 2.2.2):

TABLE 4.2

GIARDIA AND VIRUS REDUCTION CREDIT

Treatment	Giardia Credit	Virus Credit
Conventional filtration	2.5-log	2.0-log
Direct or in-line filtration	2.0-log	1.0-log
Slow sand filtration	2.0-log	2.0-log

The remainder of the log reduction has to be made up by disinfection, e.g. for systems on conventional filtration: (3-log as required for Giardia reduction) - (2.5-log Giardia credit) = (0.5-log reduction by disinfection), assuming raw water Giardia levels are less than 1 cyst/100 L.

3. T_{10} Determination (see also Section 9.2.3.7)

T_{10} values can be significantly different from calculated detention times, T, (volume/flow) and shall be determined by tracer study or by reference to typical baffling conditions. Use of the T_{10} ensures that 90% of the water will be better treated than the minimum.

TABLE 4.3

TYPICAL BAFFLING CONDITIONS**

Baffling Condition	T_{10}/T Ratio	Baffling Description
Unbaffled (mixed flow)	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities
Poor	0.3	Single or multiple unbaffled inlets and outlets, no intra-basin baffles
Average	0.5	Baffled inlet or outlet with some intra-basin baffles
Superior	0.7	Perforated inlet baffle, serpentine or perforated intra-basin baffles, outlet weir or perforated launders
Perfect (plug flow)	1.0	Very high length to width ratio (pipeline flow), perforated inlet, outlet, and intra-basin baffles

**Based on "Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems using Surface Water Sources", USEPA, October 1990.
(See Appendix C for illustration of these conditions)

4. Required CT Value

Required CT values are dependent on pH, residual concentration, temperature and the disinfectant used. The tables attached to Appendices A and B shall be used to determine the required CT.

5. Calculation and Reporting of CT Data
(See also Section 9.2.3.7)

Disinfection CT values shall be calculated daily using either the maximum hourly flow and the disinfectant residual at the same time, or by using the lowest CT value if it is calculated more frequently. Actual CT values are then compared to required CT values. Results shall be reported as a reduction Ratio, along with the appropriate pH, temperature, and disinfectant residual. The reduction Ratio must be greater than 1.0 to be acceptable. Users may also calculate and record actual log reductions.

$$\text{Reduction Ratio} = \frac{CT_{actual}}{CT_{required}}$$

6. Additive CT Values

CT values can be calculated for process units in series by calculating the reduction ratio for each step and then adding the separate reduction ratios together. Different disinfectants can also be used in different process steps.

$$\text{Reduction Ratio (Total)} = \sum \frac{CT_{actual}}{CT_{required}}$$

7. Particle Count or Microscopic Analysis Credits

Additional credits may be obtained for protozoan (Giardia, Cryptosporidium) removal by use of Particle Counting data in the 2-15 µm size range, or by use of Microscopic Particulate Analysis Data. Any additional removals from raw water to treated water measured in excess of the normal treatment credit (e.g. 2.5-log) may be added to the daily total log removal. Particle Count reductions cannot however completely replace disinfection requirements; the virus reduction component still has to be met.

8. Future CT Requirements

Future requirements may also include particle removal and/or disinfection conditions for reduction in Cryptosporidium levels.

4.1.5.2 Chlorination Equipment Requirements

For all water treatment facilities, chlorine gas under pressure shall not be permitted outside the chlorine room. Chlorine room is the room where chlorine gas cylinders and/or ton containers are stored. Vacuum regulators shall also be located inside the chlorine room. The chlorinator, which is the mechanical gas proportioning equipment, may or may not be located inside the chlorine room.

For new and upgraded facilities, from the chlorine room, chlorine gas vacuum lines should be run as close to the point of solution application as possible. Injectors should be located to minimize the length of pressurized chlorine solution lines. A gas pressure relief system shall be included in the gas vacuum line between the vacuum regulator(s) and the chlorinator(s) to ensure that pressurized chlorine gas does not enter the gas vacuum lines leaving the chlorine room. The gas pressure relief system shall vent pressurized gas to the atmosphere at a location that is not hazardous to plant personnel; vent line should be run in such a manner that moisture collecting traps are avoided. The vacuum regulating valve(s) shall have positive shutdown in the event of a break in the downstream vacuum lines.

As an alternative to chlorine gas, it is permissible to use hypochlorite with positive displacement pumping. Anti-siphon valves shall be incorporated in the pump heads or in the discharge piping.

1. Capacity

The chlorinator shall have the capacity to dose enough chlorine to overcome the demand and maintain the required concentration of the "free" or "combined" chlorine.

2. Methods of Control

Chlorine feed system shall be automatic proportional controlled, or automatic residual controlled, or compound loop controlled. In the automatic proportional controlled system, the equipment adjusts the chlorine feed rate automatically in accordance with the flow changes to provide a constant pre-established dosage for all rates of flow. In the automatic residual controlled system, the chlorine feeder is used in conjunction with a chlorine residual analyzer which controls the feed rate of the chlorine feeders to maintain a particular residual in the treated water. In the compound loop control system, the feed rate of the chlorinator is controlled by a flow proportional signal and a residual analyzer signal to maintain a particular chlorine residual in the water.

Manual chlorine feed system may be installed for groundwater systems with constant flow rate.

3. Standby Provision

As a safeguard against malfunction and/or shut-down, standby chlorination equipment having the capacity to replace the largest unit shall be provided. For uninterrupted chlorination, gas chlorinators shall be equipped with an automatic changeover system. In addition, spare parts shall be available for all chlorinators.

4. Weigh Scales

Scales for weighing cylinders shall be provided at all plants using chlorine gas to permit an accurate reading of total daily weight of chlorine used. At large plants, scales of the recording and indicating type are recommended. As a minimum, a platform scale shall be provided. Scales shall be of corrosion-resistant material.

5. Securing Cylinders

All chlorine cylinders shall be securely positioned to safeguard against movement. Tonne containers may not be stacked.

6. Chlorine Leak Detection

Automatic chlorine leak detection and related alarm equipment shall be installed at all water treatment plants using chlorine gas. Leak detection shall be provided for the chlorine rooms. Chlorine leak detection equipment should be connected to a remote audible and visual alarm system and checked on a regular basis to verify proper operation. Leak detection equipment shall not automatically activate the chlorine room ventilation system in such a manner as to discharge chlorine gas. During an emergency if the chlorine room is unoccupied, the chlorine gas leakage shall be contained within the chlorine room itself in order to facilitate a proper method of clean-up.

Consideration should also be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking one-ton cylinders where such cylinders are in use.

Chlorine leak detection equipment may not be required for very small chlorine rooms with an exterior door (e.g., floor area less than $3m^2$).

7. Safety Equipment

The facility shall be provided with personnel safety equipment to include the following: Respiratory equipment; safety shower, eyewash; gloves; eye protection; protective clothing; cyclinder and/or tonner repair kits.

Respiratory equipment shall be provided which has been approved under the Occupational Health and Safety Act, General Safety Regulation - Selection of Respiratory Protective Equipment. Equipment shall be in close proximity to the access door(s) of the chlorine room.

4.1.5.3 Chlorine Room Design Requirements

Where gas chlorination is practiced, the gas cylinders and/or the ton containers up to the vacuum regulators shall be housed in a gas-tight, well illuminated, corrosion resistant and mechanically ventilated enclosure. The chlorinator may or may not be located inside the chlorine room. The chlorine room shall be located at the ground floor level.

1. Ventilation

Gas chlorine rooms shall have entirely separate exhaust ventilation systems capable of delivering one (1) complete air change per minute during periods of chlorine room occupancy only - there shall be no continuous ventilation. The air outlet from the room shall be 150 mm above the floor and the point of discharge located to preclude contamination of air inlets to buildings or areas used by people. The vents to the outside shall have insect screens. Air inlets should be louvred near the ceiling, the air being of such temperature as to not adversely affect the chlorination equipment. Separate switches for fans and lights shall be outside the room at all entrance or viewing points, and a clear wire-reinforced glass window shall be installed in such a manner as to allow the operator to inspect from the outside of the room.

2. Heating

Chlorine rooms shall have separate heating systems, if forced air system is used to heat the building. Hot water heating system for the building will negate the need for a separate heating system for the chlorine room. The heat should be controlled at approximately 15^{o}C. Cylinders or containers shall be protected to ensure that the chlorine maintains its gaseous state when entering the chlorinator.

3. Access

All access to the chlorine room shall only be from the exterior of the building. Visual inspection of the chlorination equipment from inside may be provided by the installation of glass window(s) in the walls of the chlorine room. Windows should be at least 0.20 m^2 in area, and be made of clear wire reinforced glass.

There should also be a 'panic bar' on the inside of the chlorine room door for emergency exit.

4. Storage of Chlorine Cylinders

If necessary, a separate storage room may be provided to simply store the chlorine gas cylinders, with no connection to the line. The chlorine cylinder storage room shall have access either to the chlorine room or from the plant exterior, and arranged to prevent the uncontrolled release of spilled gas. Chlorine gas storage room shall have provision for ventilation at thirty air changes per hour. Viewing glass windows and panic button on the inside of door should also be provided.

In very large facilities, entry into the chlorine rooms may be through a vestibule from outside.

5. Scrubbers

For facilities located within residential or densely populated areas, consideration shall be given to provide scrubbers for the chlorine room.

4.1.5.4 Alternate Disinfectants

1. Chloramine

Chloramine is a very weak disinfectant for Giardia and virus reduction; it is recommended that it be used in conjunction with a stronger disinfectant. It is best utilized as a stable distribution system disinfectant.

In the production of chloramines, the ammonia residuals in the finished water, when fed in excess of stoichiometric amount needed, should be limited to inhibit growth of nitrifying bacteria.

2. Chlorine Dioxide

Chlorine dioxide may be used for either taste and odour control or as a pre-disinfectant. Total residual oxidants (including chlorine dioxide and chlorite, but excluding chlorate) shall not exceed 0.30 mg/L during normal operation or 0.50 mg/L (including chlorine dioxide, chlorite and chlorate) during periods of extreme variations in the raw water supply.

Chlorine dioxide provides good Giardia and virus protection but its use is limited by the restriction on the maximum residual of 0.5 mg/L ClO_2/chlorite/chlorate allowed in finished water. This limits usable residuals of chlorine dioxide at the end of a process unit to less than 0.5 mg/L.

Where chlorine dioxide is approved for use as an oxidant, the preferred method of generation is to entrain chlorine gas into a packed reaction chamber with a 25% aqueous solution of sodium chlorite ($NaClO_2$).

Warning: Dry sodium chlorite is explosive and can cause fires in feed equipment if leaking solutions or spills are allowed to dry out.

3. Ozone

Ozone is a very effective disinfectant for both Giardia and viruses. Ozone CT values must be determined for the ozone basin alone; an accurate T_{10} value must be obtained for the contact chamber, residual levels measured through the chamber and an average ozone residual calculated. Ozone does not provide a system residual and should be used as a primary disinfectant only in conjunction with free and/or combined chlorine.

Ozone does not produce chlorinated byproducts (such as trihalomethanes) but it may cause an increase in such byproduct formation if it is fed ahead of free chlorine; ozone may also produce its own oxygenated byproducts such as aldehydes, ketones or carboxylic acids. Any installed ozonation system must include adequate ozone leak detection alarm systems, and an ozone off-gas destruction system.

Ozone may also be used as an oxidant for removal of taste and odour or may be applied as a pre-disinfectant.

Note: If alternative disinfectants are used as the primary disinfectant, monitoring will be in accordance with section 9.2.3.7.

4.2 Groundwater - Well Development and Construction

4.2.1 Minimum Requirements

As a minimum standard, well construction shall conform to the requirements of the Water Well Regulation.

Adequate geological, hydrological, and raw water quality information shall be obtained and submitted to AEP as part of the approval process prior to construction. In particular, the engineering study should address several factors including the safe well yield, the potential sources of pollution, and the water treatment requirements.

4.3 Treatment of Groundwater

4.3.1 Minimum Treatment Requirements

Minimum treatment for groundwater shall be disinfection if the waterworks system it supplies has 15 or more service connections or has 3 or more kilometres of water distribution system, unless otherwise exempted by AEP in accordance with Section 2.3 - Minimum Performance Requirements for Treatment Components (Groundwater).

4.3.2 Removal of Fluoride

Naturally occurring fluoride up to a concentration of 2.4 mg/L is acceptable. Raw water with concentration greater than 2.4 mg/L shall be treated to reduce the level to 0.8 mg/L, which is the optimum level for the control of dental caries.

4.3.3 Disinfection

Performance requirements for treatment with respect to disinfection of groundwaters are stipulated in Section 2.0 - Performance Standards - Waterworks System.

Design standards for disinfection facility are the same as for surface water, as outlined in Sections 4.1.5.2 and 4.1.5.3.

4.4 Water Treatment Chemicals

4.4.1 Labels and Material Safety Data Sheets

Federal and provincial legislation requires hazardous products at worksites to be labelled and information be made available to workers through Material Safety Data Sheets. It also requires employers to train workers and workers to be knowledgeable with the Workplace Hazardous Materials Information System (WHMIS).

Most chemicals used for water treatment are "controlled products". "Controlled products", hazardous products at the worksite that meet certain criteria, will either be labelled with a supplier label or worksite label, in accordance with the requirements specified in the latest edition of WHMIS.

For appropriate use of the chemical, water treatment operators should be aware of the chemical purity (concentration), shelf life, expiry data, maximum dosage and use restrictions. This information can usually be found on the supplier labels but may be added to the worksite label for ease in use.

More specific information on the hazardous ingredients, hazards, health and safety risks, safe handling instructions, emergency and first aid measures are contained on a Material Safety Data Sheet (MSDS). MSDS obtained from the supplier and not more than three years old must be available at the worksite for all "controlled products" unless it is laboratory product where the label may contain all the information required on a MSDS.

Storage buildings and outside storage tanks should be labelled or placarded with the name of the product and/or hazards of the products.

4.4.2 Storage and Handling

4.4.2.1 General Provisions

Storage should be provided for at least thirty (30) days of consumption at the maximum anticipated chemical usage rate, allowing for variations in chemical dosage and flow in that period. Storage capacity for essential chemicals should be at least sixty (60) days, with ninety (90) days preferred. Where deliveries of chemicals may be interrupted by adverse weather conditions in isolated locations, provision should be made for increased storage capacity. Where deliveries at short notice can be assured and the material is not essential to the production of safe water, then requirements may be reduced. If practical, there should also be sufficient storage space to accommodate 'full load' deliveries.

Chemical storage areas should be segregated from the main areas of the treatment plant, with separate storage areas provided for each chemical. Where chemicals in storage may react dangerously with other materials in storage, e.g. chlorine and ammonia or strong acids and alkalis, segregated storage shall be provided. The storage and feed equipment areas should be arranged for convenience of operation and observation, and located to provide easy access for chemical deliveries.

It is strongly recommended that all chemical storage be at or above the surrounding grade. Where subsurface locations for chemical storage tanks are proposed, these locations shall be free from sources of possible contamination, having drainage for ground waters, chemical spills, and overflows. Where above grade storage is provided, due consideration should be given to the method of unloading chemicals. In general, storage areas should be arranged to prevent any chemical spills. Floor surfaces should be smooth and impervious, slip-proof, and sloped so as to drain rapidly; walls and floors should be protected with a chemical-resistant finish. Storage areas should have eye-wash and/or deluge shower facilities, adequate facilities for cleaning up chemical spills, space for cleaning and storage of the recommended protective equipment, and adequate warning signs to identify hazards. It is recommended that all doors in chemical buildings open outward, and that corridors or space between storage areas be a minimum of 1.5 m wide to permit the safe movement of materials.

Chemical ventilation systems should be arranged so that air is exhausted outside the building and also so that slight negative pressures are maintained where dry chemicals are in use, as a dust control measure. Where large amounts of dust are anticipated, appropriate local exhaust systems and filters or scrubbers should be provided in the ventilation system. Ventilation systems should be designed specifically for corrosive service, and special measures taken in dust systems to prevent build-up of static or other explosive conditions.

4.4.2.2 Liquid Chemicals

All bulk storage tanks should have an adequately sized fill line of at least 50 mm diameter, sloped to drain into the tank. The fill line should be adequately identified at the end that is remote from the tank, and provision should be made to drain this fill line if required.

Each tank should have an adequate vent line, minimum size 50 mm with a downturned end. Where venting outside the room is required, the vent should be provided with an insect screen.

All tanks should have an overflow which is adequate for the rate of fill proposed for the tank, sloped down from the tank with a downturned end with free discharge located where it can be readily noticed. Overflow shall not be directly discharged into storm drainage system or to a watercourse. If discharged into the sanitary sewer system, the overflow pipes shall not connect directly to the sewer, and where they pass into a receiving sump or conduit they shall terminate at least two pipe diameters above the maximum level in the sump. Each tank should also be provided with means to indicate the level of contents in the tank, and where an external level gauge is provided a shut-off valve at the tank connection is recommended.

All storage tanks should also be surrounded by a structure to contain the volume of the largest tank in the event of a rupture or spill.

In small treatment plants, day-tanks shall be provided for the liquid chemicals.

4.4.2.3 Dry Chemicals

Where dry chemicals are to be used, provision should be made to minimize handling and dust problems. From a materials handling perspective, granular materials should be preferred to powders.

Particular care should be taken to protect workers and mechanical and electrical equipment from fine dust. Where exhaust fans, filters, and conveying systems are used, grounding shall be provided to prevent the build-up of static electricity. Dust control equipment should also be protected against moisture accumulation.

Bulk storage silos should be provided with adequately sized fill openings, and fill lines where necessary should be smooth internally with long radius elbows. Silos should have suitable level indicating devices and should be equipped with a pressure relief valve when pneumatic fill systems are provided.

If powdered activated carbon (PAC) is used, spark-free lighting and electrical systems shall be provided. Provision shall also be made to scrub or filter the carrier air, when dry PAC is off-loaded into silos.

4.5 Fluoridation

4.5.1 General Requirements

In addition to the specific criteria outlined in this Section, all requirements specified under Section 4.4 - Water Treatment Chemicals shall also be complied if fluoridation is practiced.

Any person proposing to add fluoride to a potable water supply shall apply for and obtain an approval or an amendment to an approval from AEP. This application shall contain the following information:

1. a copy of the bylaw of the municipal council which provides the authority to fluoridate;

2. the number of the appropriate approval issued with respect to the existing municipal plant; and,

3. an engineering report, including

   (i) a description of the proposed fluoridation equipment,

   (ii) a statement identifying the fluoride compound that is proposed to be added,

   (iii) a description of chemical storage and ventilation,

   (iv) a description of the water metering used at the water treatment plant,

   (v) the generic name of the chemical to be used as the source of fluoride ion, and its fluoride content,

   (vi) a current chemical analysis of the fluoride content of the raw water,

   (vii) the name and qualifications of the person directly responsible for the operation of the proposed fluoridation process,

   (viii) the type of equipment proposed at the water treatment plant to determine the fluoride concentration of the water, and

   (ix) the description of the testing procedure to be used to determine the fluoride level of the water.

4.5.2 Chemical Feed

The equipment used for feeding the fluoride to water shall be accurately calibrated before being placed in operation, and at all times shall be capable of maintaining a rate of feed within 5% of the rate at which the machine is set.

The following chemical feed practices apply:

1. Where a dry feeder of the volumetric or gravimetric type is used, a suitable weighing mechanism shall be provided to check the daily amount of chemical feed;

2. Hoppers should be designed to hold a 24 hour supply of the fluoride compound and designed such that the dust hazard to operators is minimized;

3. Vacuum dust filters shall be installed with the hoppers to prevent dust from rising into the room when the hopper is filled;

4. Dissolving chambers are required for use with dry feeders, and the dissolving chambers shall be designed such that at the required rate of feed of the chemical the solution strength will not be greater than 1/4 of that of a saturated solution at the temperature of the dissolving water. The construction material of the dissolving chamber and associated piping shall be compatible with the fluoride solution to be fed;

5. Solution feeders shall be of the positive displacement type and constructed of material compatible with the fluoride solution being fed;

6. The weight of the daily amount of fluoride fed to water shall be accurately determined;

7. Feeders shall be provided with anti-syphon valves on the discharge side. Wherever possible, positive anti-syphon breakers other than valves shall be provided;

8. A "day tank" capable of holding a 24 hour supply of solution should be provided;

9. All equipment shall be sized such that it will be operated in the 20 to 80 percent range of the scale, and be capable of feeding over the entire pumpage range of the plant;

10. Alarm signals are recommended to detect faulty operation of equipment; and,

11. The fluoride solution should be added to the water supply at a point where the fluoride will not be removed by any following treatment processes and where it will be mixed with the water. It is undesirable to inject the fluoride compound or solution directly on-line unless there are provisions for adequate mixing.

4.5.3 Metering

Metering of the total water to be fluoridated shall be provided, and the operation of the feeding equipment is to be controlled unless specifically exempted by AEP. Control of the feed rate shall be automatic/ proportional controlled, whereby the fluoride feed rate is automatically adjusted in accordance with the flow changes to provide a constant pre-established dosage for all rates of flow, or (2) automatic/ residual controlled, whereby a continuous automatic fluoride analyzer determines the residual fluoride level and adjusts the rate of feed accordingly, or compound loop controlled, whereby the feed rate is controlled by a flow proportional signal and residual analyzer signal to maintain a constant residual.

4.5.4 Alternate Compounds

Any one of the following fluoride compounds may be used:

1. Hydrofluosilicic acid;
2. Sodium fluoride; or,
3. Sodium silicofluoride.

Other fluoride compounds may be used if approved by AEP.

4.5.5 Chemical Storage and Ventilation

The fluoride chemicals shall be stored separately from other chemicals, and the storage area shall be marked "FLUORIDE CHEMICALS ONLY". The storage area should be in close proximity to the feeder, kept relatively dry, and provided with pallets, if using bagged chemical, to allow circulation of air and to keep the containers off the floor.

4.5.6 Record of Performance

Accurate daily records shall be kept. These records shall include:

1. the daily reading of the water meter which controls the fluoridation equipment or that which determines the amount of water to which the fluoride is added;
2. the daily volume of water fluoridated;
3. the daily weight of fluoride compound in the feeder;
4. the daily weight of fluoride compound in stock;
5. the daily weight of the fluoride compound fed to the water; and,
6. the fluoride content of the raw and fluoridated water determined by laboratory analysis, with the frequency of measurement as follows:

    (i) treated water being analyzed continuously or once daily, and

    (ii) raw water being analyzed at least once a week.

4.5.7 Sampling

In keeping the fluoride records outlined in Section 4.5.6, the following sampling procedures are required:

1. A sample of raw water and a sample of treated water shall be forwarded to an approved independent laboratory for fluoride analysis once a month;

2. On new installations or during start-ups of existing installations, weekly samples of raw and treated water for a period of not less than four consecutive weeks (or for a period as required by AEP) shall be submitted to either AEP or other designated laboratory to determine the fluoride concentration; and,

3. In addition to the reports required, AEP may require other information that is deemed necessary.

4.5.8 Safety

The following safety procedures shall be maintained:

1. All equipment shall be maintained at a high standard of efficiency, and all areas and appliances shall be kept clean and free of dust. Wet or damp cleaning methods shall be employed wherever practicable;

2. Personal protective equipment shall be used during the clean-up, and appropriate covers shall be maintained over all fluoride solutions;

3. At all installations, safety features are to be considered and the necessary controls built into the installation to prevent an overdose of fluoride in the water. This shall be done either by use of day tanks or containers, anti-syphon devices, over-riding flow switches, sizing of pump and feeders, determining the length and duration of impulses, or other similar safety devices as approved by AEP;

4. Safety features shall also be provided to prevent spills and overflows as determined by AEP.

5. Individual dust respirators, chemical safety face shields, rubber gloves, and protective clothing shall be worn by all personnel when handling or being exposed to the fluoride dust;

6. Chemical respirators, rubber gloves, boots, chemical safety goggles and acid proof aprons shall be worn where acids are handled;

7. After use, all equipment shall be thoroughly cleaned and stored in an area free of fluoride dusts. Rubber articles shall be washed in water, and hands shall be washed after the equipment is stored; and,

8. All protective devices, whether for routine or emergency use, shall be inspected periodically and maintained in good operating condition.

4.5.9 Repair and Maintenance

Upon notifying AEP and the appropriate local board of health, a fluoridation program may be discontinued when necessary to repair or replace equipment, but shall be placed in operation immediately after the repair or replacement is complete. Records shall be maintained and submitted during the period that the equipment is not in operation.

4.6 Treatment Plant Waste - Handling and Disposal

Provision shall be made for proper treatment and/or disposal of all water treatment plant wastes. These include sanitary wastes, filter backwash, filter-to-waste and sludges.

4.6.1 Sanitary Wastes

All sanitary wastes from water treatment plants shall be handled by direct discharge to a sanitary sewer system or to an approved wastewater treatment facility. These sanitary wastes should be kept separate from other process wastes to avoid the need to treat all plant wastes in the same manner as the sanitary wastes.

The practice of passing wastewater lines through potable water reservoirs shall not be allowed.

4.6.2 Filter Backwash

Backwash waste may be discharged directly to a sanitary sewer system, if the sewers and the wastewater treatment plant can withstand the hydraulic surges.

Backwash waste may not be discharged directly to an open body of water. Exceptions may be made only if it can be demonstrated that there are no significant adverse effects on the receiving body of water. Based on the quantity and quality of backwash waste and the sensitivity of the receiving body of water, AEP may request for an impact assessment study to ascertain the need for backwash waste treatment before discharging to the environment.

Discharge of backwash water into a raw water reservoir will not be permitted. Recycling of treated backwash waste after treatment, though not a good practice, may be acceptable depending on the treatment method that will be used to treat the backwash water. The treatment method will be approved in conjunction with the policy and procedures outlined in Section 5.2 - Unproven or Innovative/Alternative Technologies of the Guiding Principles and Policy Document (1996).

4.6.3 Filter-to-Waste

Filter-to-waste may be discharged directly to a sanitary sewer system, if the sewers and the wastewater treatment plant can withstand the hydraulic surges. Filter-to-waste may also be recycled to the pre-treatment works or to the raw water reservoir.

4.6.4 Sludges

Sludges generated at water treatment plants shall be treated and handled in a manner approved by AEP. The following sections deal with the various coagulant and softening sludges, and also outline the alternative methods of treatment and disposal.

4.6.4.1 Aluminum Sludge

Aluminum sludges are produced when aluminum sulphate or poly aluminum chloride is used for coagulation and flocculation of raw water supplies to remove turbidity and/or colour. The floc particles are removed mainly during the sedimentation process in which the flocs slowly settle and form a sludge blanket. This blanket (referred to as aluminum sludge) consists of aluminum hydroxide and other particles and flocculated materials. To maintain adequate efficiency of sedimentation basins, the aluminum sludge requires periodic or continuous removal and disposal.

Approval for the disposal of aluminum sludge shall be obtained on a site-specific basis. The following are a number of alternative methods of handling and disposing of aluminum sludge from water treatment plants:

1. Direct discharge to a wastewater treatment plant or sanitary sewer. Consideration should be given to the potential beneficial and adverse effects on the wastewater treatment facility;

2. Lagooning. Lagoons can be used as permanent storage facilities, long-detention settling lagoons to provide freeze/thaw cycle with supernatant disposal, or drying beds using evaporation;

3. Mechanical thickening and dewatering. Once thickened and dewatered to approximately 20% solids, sludge can be placed at an approved disposal site, usually a landfill site used exclusively for sludge;

4. Direct discharge to a stream. This option shall be approved only where there is a negligible environmental impact and it has been demonstrated that the aesthetics and downstream water users will not be affected. This option should only be considered if alternate management options are unavailable;

5. Land disposal. Land disposal to a sanitary landfill site or agricultural land of dilute or thickened and dewatered sludge is potentially harmful and shall be thoroughly reviewed with the approving authority prior to implementation;

6. Alum recovery. Alum recovery and re-use at water treatment plants is not considered a viable option; concerns with recovery costs and recycle of organics and heavy metals are the major reasons for rejection of this sludge handling alternative. It may, however, be feasible to recover water treatment plant alum sludge for re-use at a wastewater treatment plant utilizing chemical precipitation of phosphorus; and,

7. Reduction of sludge quantity. A reduction in the quantity of solids is possible by utilizing a number of management practices including presedimentation by raw water storage, the use of polymer, or the effective design and operation of the coagulation/flocculation facilities.

4.6.4.2 Lime Sludge

Lime in the form of calcium hydroxide is commonly used as a coagulant or softening agent. Handling and disposal methods are similar to those for alum sludge (see Section 4.6.4.1), however under no circumstances may lime sludge be discharged directly to any watercourse in Alberta. Application shall be made to the approving authority for disposal of lime sludge on a case specific basis.

Because of potential deposition problems, the practice of lime sludge disposal to sewers is not recommended.

4.6.4.3 Sludges from Other Coagulants

There are a number of coagulants (such as ferric/ferrous compounds, polymers, polyelectrolytes, sodium aluminate), which are an alternative to alum and which may have better sludge dewatering characteristics. The handling and disposal of these other chemical sludges shall be approved by AEP on a site specific basis.

4.7 Transmission and Distribution Mains and Reservoirs

4.7.1 Pipe Performance Standards

Where pipe performance standards exist, all materials that are used in the construction of the plant, transmission and distribution systems shall meet or exceed AWWA and/or CSA standards.

4.7.2 Frost Protection for Mains and Reservoirs

4.7.2.1 Mains

To prevent freezing and damage due to frost, pipes shall have a minimum cover above the crown of the pipe the greater of:

1. 2.5 m, or

2. the depth of frost penetration for the location based on the coldest three years during the past 30 years, or, where this period of record is not available, the coldest year during the past 10 years with an appropriate safety factor.

Where these minimum frost protective covers can not be achieved, AEP may allow an exemption if the owner can demonstrate incorporation of appropriate special precautions in the selection of pipe, bedding and insulation material.

4.7.2.2 Reservoirs

Reservoirs and appurtenances (such as overflows and vents) shall be designed to prevent damage from freezing. Elevated tanks and standpipes shall be insulated and hot water recirculated, or heat traced, to prevent problems associated with ice formation. Generally, they should be maintained at a temperature of 4^{0}C or greater.

4.7.3 Cross-Connection Controls

There shall be no physical connection between any waterwaste systems and a sanitary or storm sewer which may allow the passage of wastewater into the potable water supply.

The following cross-connection controls are necessary to preclude the entrance of contaminants into the water distribution system.

4.7.3.1 Horizontal Separation of Watermains and Sewers

A watermain is defined as a pipeline that conveys water and forms an integral part of the water distribution system as defined in EPEA.

Unless otherwise approved by AEP, the minimum horizontal separation between a watermain and a storm or a sanitary sewer or manhole shall be 2.5 m, the distance being measured centre to centre.

Unusual conditions including excessive rock, dewatering problems, or congestion with other utilities may prevent the normal required horizontal separation of 2.5 m. Under these condition(s), AEP may approve a lesser separation distance, provided that the crown of the sewer pipe is at least 0.5 m below the watermain invert.

Where extreme conditions prevent the 2.5 m separation and vertical separation cannot be obtained, the sewer shall be constructed of pipe and joint materials which are equivalent to watermain standards.

Under no circumstances shall the horizontal separation be less than 1.0 m.

4.7.3.2 Pipe Crossings

Under normal conditions, watermains shall cross above sewers with a sufficient vertical separation to allow for proper bedding and structural support of the water and sewer mains.

Where it is necessary for the watermain to cross below the sewer, the watermain shall be protected by providing:

(a) A vertical separation of at least 0.5 m from watermain crown to sewer invert;

(b) Structural support of the sewer to prevent excessive joint deflection and settling; and

(c) A centering of the length of watermain at the point of crossing so that the joints are equidistant from the sewer.

4.7.3.3 Valve, Air Relief, Meter and Blow-Off Chambers

Chambers or pits containing valves, blow-offs, meters, or other water distribution appurtenances shall not be directly connected to a storm or sanitary sewer, nor shall blow-offs or air relief valves be directly connected to any sewer.

4.7.3.4 Backflow Prevention and Control

Backflow preventers shall be installed at any location where a connection is made to an approved waterworks system for the purpose of serving a hamlet, municipal development, privately owned development, a truckfill station or other waterworks system located outside the service boundary of the approved waterworks system. Backflow preventers shall be installed in accordance with the latest edition of the Cross Connection Control Manual, published by AWWA (Western Canada).

4.7.4 Disinfection of Mains and Reservoirs

4.7.4.1 Mains

All new or repaired watermains shall be disinfected according to the current American Water Works Association (AWWA) Standard for Disinfecting Water Mains. New lines shall be thoroughly flushed and chlorinated at a dosage of 50 mg/L for 12 hours. In short lines and if portable chlorination equipment is not available, thorough flushing and maintenance of a free chlorine residual of 1.0 mg/L after 24 hours shall be carried out, with a test for residual chlorine being made at the end of the test period.

In repairing breaks, care shall be taken to exclude dirt and ditch water. The section shall be thoroughly flushed and should remain filled with water having a free chlorine residual of 1.0 mg/L after a one hour period before being placed in service. This should be followed by a bacteriological test to verify the potability of the water. Following disinfection of all new watermains bacteriological quality shall be assessed to demonstrate acceptable quality of water before putting the mains in service.

4.7.4.2 Reservoirs

Treated water storage reservoirs shall also be disinfected and flushed before being put into service, in accordance with the current AWWA Standard.

4.7.4.3 Discharge of Superchlorinated Water

Chlorinated water used for disinfection of mains and reservoirs shall not be directly drained into the storm sewer or into an open body of water; dechlorination would be required before being discharged into the environment.

4.8 Health and Safety Act

The design and construction of all components of the waterworks system shall conform to the safety provisions of the Alberta Occupational Health and Safety Act and Regulations.

5.0 DESIGN STANDARDS - WASTEWATER SYSTEM

5.1 Wastewater Collection

5.1.1 Sanitary Sewers

5.1.1.1 Minimum Pipe Diameter

Minimum pipe diameter for gravity sewer, in general, shall be 200 mm in diameter. However, under limited circumstances, sewers of not less than 150 mm diameter may be allowed if the owner can demonstrate that a 150 mm diameter sewer is adequate and will not be detrimental to the operation and maintenance of the sewer system.

The hydraulic capacity of a gravity sewer should be based on consideration of factors such as projected in-service roughness coefficient, slope, pipe material and actual in-service flows. In general, sewers larger than the minimum size required shall be chosen so that the minimum velocity at the average flow is not less than 0.6 m/s for self cleansing purposes, and the maximum velocity at the peak design flow is not greater than 3.0 m/s to minimize turbulence and erosion. Under exceptional circumstances, where velocities greater than 3.0 m/s are attained, provision shall be made to protect against displacement by erosion and impact.

For small diameter low pressure or vacuum sewer collection systems, the designer shall provide hydraulic calculations and/or supporting information to verify the proposal.

5.1.1.2 Minimum Pipe Slope

All gravity sewers between manholes shall be laid with uniform slopes equal to or greater than the minimum slopes outlined in Table 5.1.

These minimum slopes are not based on an assumed specific pipe roughness coefficient, but rather on historical satisfactory operation of sewers meeting or exceeding these slopes under varying flow conditions. Under special conditions, slopes slightly less than those shown may be permitted. If the proposed slope is less than the minimum slope of the smallest pipe which can accommodate the peak wastewater design flow, the actual depths and velocities at average, and peak wastewater design flow for each design section of the sewer shall be calculated by the designer and submitted to AEP. (See Sections 5.1.1.3 and 5.1.1.4).

For new construction, the pipe slope shall be determined using the minimum pipe diameter necessary for the design volume of wastewater. Further, a manhole outlet pipe diameter shall not be reduced to be smaller than the inlet pipe diameter to compensate for increased slope in the outlet line. In retrofit situations, where the minimum slope cannot be achieved due to site constraints, lower than the minimum slopes may be allowed. Under this situation, the owners shall make a commitment to undertake additional operation and maintenance measures to prevent solids deposition in the line.

5.1.1.3 Minimum Flow Depths

As per Section 5.1.1.2, slopes slightly less than those recommended for the 0.6 m/s velocity, when flowing full, may be permitted. Such decreased slopes will only be considered where the depth of flow will be 0.3 of the diameter or greater for average design flow. The owner of the sewer system shall give written assurance to AEP that any additional sewer maintenance required as a result of reduced slopes will be provided.

5.1.1.4 Solids Deposition

The pipe diameter and slope shall be selected to obtain practical velocities to minimize settling problems. As per Section 5.1.1.2, oversize sewers will not be approved on new constructions, to justify using flatter slopes. If the proposed slope is less than the minimum slope of the smallest pipe which can accommodate the peak wastewater design flow, the actual depths and velocities at average, and peak wastewater design flow for each design selection of the sewer shall be calculated by the designer and submitted to AEP.

5.1.1.5 Alignment

Curvilinear alignment of sewers may be allowed in the design of the collection system. Where curved sewers are used, the designer shall not exceed the maximum angle at which the joints remain tight. Curved sewers shall be laid with a radius of at least 60 m unless otherwise supported by manufacturer's specifications.

The minimum slopes for curved sewers shall be 50 percent greater than the minimum slopes required for straight runs; this requirement will be waived if the designer submits calculations to demonstrate that increased slope is not required to achieve self-cleansing velocity.

5.1.1.6 Frost Protection

Frost protection criteria for sewers is the same as for water mains. (See section 4.7.2.1 for details)

5.1.1.7 Cross Connections

Cross connection prevention is the same as for water supply. (See Section 4.7.3 for details).

TABLE 5.1

MINIMUM DESIGN SLOPES FOR SANITARY SEWERS

Sewer Diameter (mm)	Minimum Design Slope (m/100 m)
200	0.40
250	0.28
300	0.22
375	0.15
450	0.12
525	0.10
600	0.08

5.1.2 Wastewater Pump Stations

5.1.2.1 Site Constraints

Wastewater pump station structures and electrical and mechanical equipment should be protected from physical damage by the 100 year flood. Wastewater pump stations should remain fully operational and accessible during the 25 year flood.

5.1.2.2 Pumps

Wastewater pump station shall be designed with multiple pump units. Where only two units are provided, they shall be of the same size. Units shall have capacity such that, with any unit out of service, the remaining units will have capacity to handle the peak wastewater design flow.

5.1.2.3 Safety Ventilation

1. General

Adequate ventilation shall be provided for all pump stations. Ventilation systems, including fresh air intake louvres/openings shall be designed to function year round; screen openings should be sized to avoid build-up of frost during winter to prevent subsequent blockages. Where the dry well is below the ground surface, mechanical ventilation is required. If screens or mechanical equipment requiring maintenance or inspection are located in the wet well, permanently installed ventilation is required. There shall be no interconnection between the wet well and dry well ventilation systems.

Where ventilation of pump stations results in odour problems/complaints, the owner shall take steps to control the odour in accordance with section 7.3.3.

2. Air Inlets and Outlets

In dry wells over 5.0 m deep, multiple inlets and outlets are desirable. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions in air ducts should be avoided to prevent clogging.

In dry wells under 5.0 m deep, the fresh air shall be forced into the well at a point 150 mm above the pump floor and allowed to escape through vents in the roof; in wet wells, the fresh air shall be forced into the well at a point 150 mm above the high water level and allowed to escape through vents to the atmosphere.

3. Electrical Controls

Switches for operation of ventilation equipment shall be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. Consideration should be given also to automatic controls where intermittent operation is used. The manual lighting ventilation switch shall override the automatic controls.

4. Fans, Heating and Dehumidification

The fan wheel shall be fabricated from non-sparking material. Automatic heating and dehumidification equipment shall be provided in all dry wells.

5. Wet Wells

Wet well ventilation may be either continuous or intermittent. Ventilation, if continuous, shall provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour during the period the wet well is occupied.

Air shall be forced into the wet well by mechanical means. For continuous ventilation, to facilitate free movement of air, the wet well may be exhausted at the highest elevation level in the structure at a rate not exceeding three air changes per hour; this rate shall not be exceeded to maintain positive pressure in the well.

Portable ventilation equipment shall be provided for use at submersible pump stations and wet wells with no permanently installed ventilation equipment.

6. Dry Wells

Dry well ventilation may be either continuous or intermittent. Ventilation, if continuous, shall provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour for ten minutes before entering the dry well and six complete air changes per hour to conserve heat during the period the dry well is occupied. A system of two speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic switch over to six changes per hour may be used.

7. Monitoring

Provision should be made in the system design to verify that the ventilation fan is operational, and that air change capacity is achieved. Portable or built-in sensing equipment for measurement of hydrogen sulphide, oxygen depletion, and combustibles should also be provided.

5.1.2.4 Separation

Dry wells, including their superstructure, shall be completely separated from the wet well. Common walls must be gas tight. All penetrations, e.g. electrical conduits, through the common wall shall be sealed.

5.1.2.5 Access

Suitable and safe means of access to dry wells and to wet wells shall be provided for inspection and cleaning. The access into the wet well shall not be through the superstructure where the pumping equipment and appurtenance may be housed. Gasketed replacement plates shall be provided to cover the opening to the wet well for pump units removed for servicing. To minimize the need for entry to the wet well, valving shall not be located in the wet well.

5.1.2.6 Water Supply

There shall be no physical connection between any potable water supply and a wastewater pumping station which under any conditions might cause contamination of the potable water supply. If a potable water supply is brought to the station, it should comply with conditions stipulated under Section 5.2.3.4 (3). In-line backflow preventers are not acceptable.

5.1.3 Screens and Grit Removal Facilities

5.1.3.1 Access and Ventilation

Screens located in pits more than 1.2 m deep shall be provided with stairway access. Access ladders, in lieu of stairways, are acceptable for pits less than 1.2 m deep.

Screening devices, installed in a building where other equipment or offices are located, shall be:

- isolated from the rest of the building,
- provided with separate outside entrances, and
- provided with separate and independent fresh air supply.

Fresh air shall be forced into enclosed screening device areas or into open pits more than 1.2 m deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required, at least six complete air changes per hour shall be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour shall be provided when workmen enter the area.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. The fan wheel should be fabricated from non-sparking material. Gas detectors shall also be provided.

For grit removal facilities housed above or below grade, ventilation shall be provided either continuously at a rate of six complete air changes per hour, or intermittently at a rate of 30 air changes per hour.

5.1.4 Health and Safety Act

The design and construction of all components of the wastewater collection system shall conform to the safety provisions of the Alberta Occupational Health and Safety Act and Regulations.

5.2 Wastewater Treatment and Disposal

5.2.1 Wastewater Lagoons

5.2.1.1 General Requirements

The minimum design standards for wastewater lagoons are a function of the average daily design flow. AEP has established acceptable cell configurations based on design flow; the following sections outline the respective component configurations required to meet the minimum standards for wastewater lagoons as specified in Section 3.0 Performance Standards - Wastewater Systems.

5.2.1.2 System Components and Configuration

1. Anaerobic Cells

Anaerobic cells shall operate at a minimum depth of 3.0 m and retain influent flow for a 2 day period based on average daily design flow.

In order to provide a cell bottom of at least 3 metres square for adequate mobility of construction equipment and sufficient bottom area for sludge accumulation, the minimum practical design volume of an anaerobic cell with 3:1 inside slopes and 3 m of operating depth is approximately 500 m^3. Therefore, the minimum average daily design flow at which an anaerobic cell is practical is approximately 250 m^3/d.

When design flows warrant the provision of anaerobic cells, there shall be a minimum of 2 and a maximum of 4 cells operated in series. The number of anaerobic cells is a function of design daily flows as illustrated in Table 5.2 and Figure 5.1.

In designing the anaerobic cells, consideration shall be given to incorporate desludging features.

2. Facultative Cells

The facultative cell shall operate at a maximum depth of 1.5 m and retain influent wastewater for at least 60 days based on average daily design flow. The purpose of this cell is to biologically stabilize the wastewater under predominantly aerobic conditions. The cell(s) follows the anaerobic treatment (in cases where anaerobic cells are warranted), precedes long-term detention storage, and is a requirement for all wastewater lagoon systems in Alberta. Refer to Figure 5.1.

TABLE 5.2

WASTEWATER LAGOON REQUIREMENTS

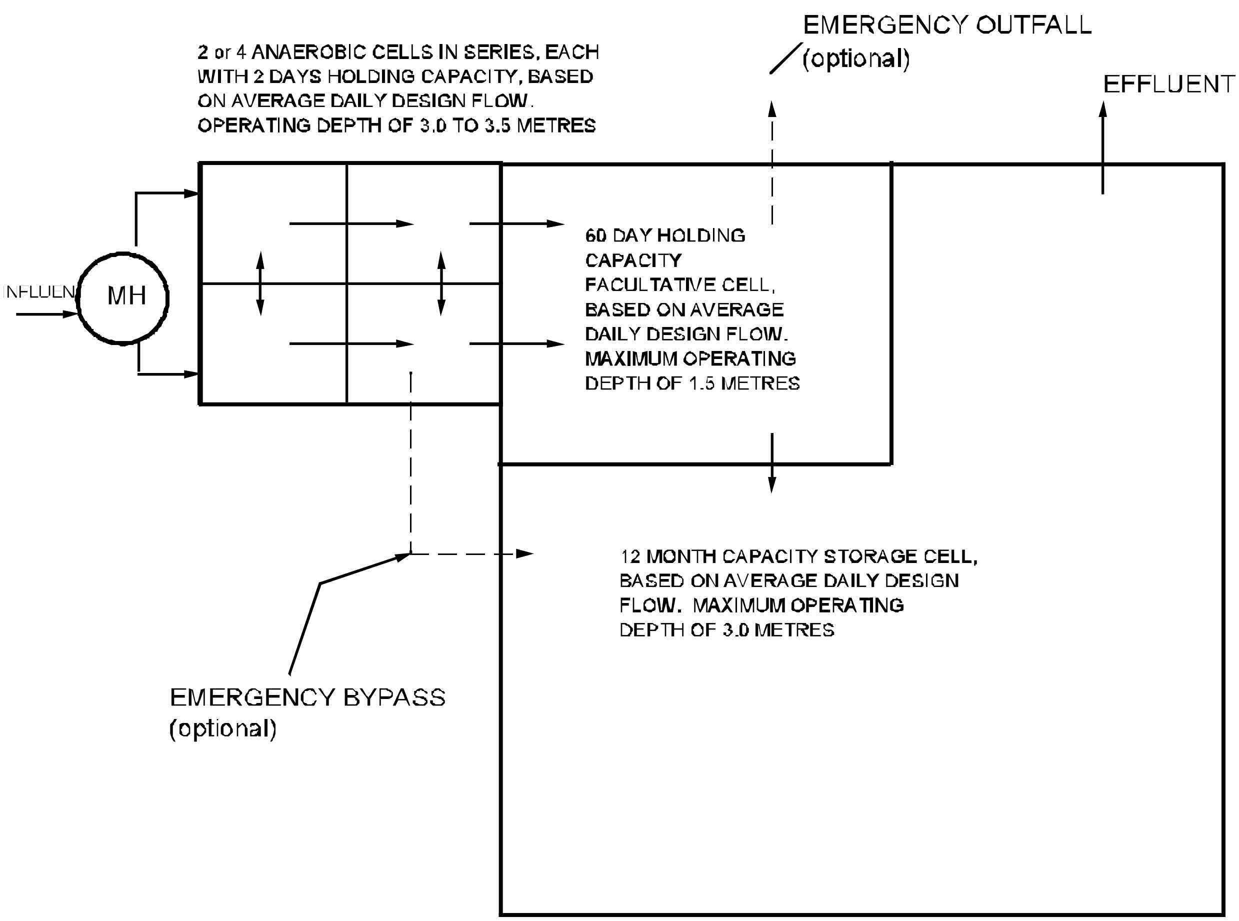
Average Daily Design Flow (m^3)	Number of Anaerobic Cells	Requirement for Facultative Cell(s)	Requirement for 12 month Storage Cell(s)
Less than 250	0 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m
250-500	2 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m
Greater than 500	4 Min. depth = 3.0 m	Yes Max. depth = 1.5 m	Yes Max. depth = 3.0 m

3. Storage Cell(s)

The storage cell(s) shall operate at a maximum depth of 3.0 m and shall retain influent wastewater for a minimum retention period of 12 months based on average daily design flows. However, if historical climatology data shows that average evaporation exceeds the average precipitation, then the net evaporation may be taken into account in sizing the storage cells. Under this scenario, the minimum "free-board" shall be increased to accommodate additional flows in an anomalous precipitation year.

The purpose of storage is to provide additional wastewater treatment (including nutrient removal) under facultative conditions, and to reduce the environmental impact on the receiving drainage course by facilitating the annual discharge of high quality effluent wastewater.

FIGURE 5.1
TYPICAL WASTEWATER LAGOONS



5.2.1.3 Design Considerations

There are a number of siting and design features which must be considered in order to protect pubic health, to maximize public safety, and also to meet the minimum standards for wastewater lagoon design.

1. Isolation of Cells

For maintenance purposes, the cells may have to be by-passed for brief periods of time. The design shall take this into account and have provisions built-in to isolate any cell.

2. Setback Distances

Setback distances from wastewater lagoons are required to buffer the effect of potential odours and to provide a margin of public safety. Setbacks also serve to protect the physical integrity of nearby buildings and roads.

Table 5.3 outlines the minimum horizontal setback distances for wastewater lagoons.

3. Site Constraints

The facility shall be protected from physical damage by the one hundred year flood. Treatment works should remain fully operational and accessible during the twenty five year flood. On a site-specific basis, it may also be necessary to provide an adequate setback distance to an adjacent drainage course.

4. Fencing

The wastewater lagoons shall be enclosed with a fence. The fence shall be designed and constructed to preclude the entrance of children and to discourage trespassing. The fence shall also serve to preclude the entrance of livestock. Where the lagoons are located near developed areas, a chain link fence may be required to preclude children from gaining entry.

Fences shall be located away from the outside toe of the berms in order to facilitate mowing and maintenance operations. In addition, an access gate shall be provided to allow entry of maintenance equipment, and this gate shall be provided with a lock to preclude entrance of unauthorized personnel.

5. Signs

Warning signs shall be provided at appropriate locations along the fenced perimeter of the wastewater lagoons. Each sign should identify the facility (by municipality or owner), advise against trespassing, and provide emergency contact phone numbers and/or addresses.

6. Access

All-weather access to the wastewater lagoons shall be provided.

7. Surface Runoff

Wastewater lagoons shall have adequate site drainage to divert surface runoff which would otherwise cause damage to the system.

TABLE 5.3

SETBACK DISTANCES FROM WASTEWATER LAGOONS

Minimum setback distance (m) from the "working area" of the wastewater lagoon to:	
The property line of the land where the lagoon is located	30
The designated right-of-way of a rural road or railway	30
The designated right-of-way of a primary or secondary highway	100
An "Occupied Building" where the lagoon serves a designated municipality	300
Any "Occupied Building" on the property of a privately owned rural development which the lagoon serves	100

***** "Occupied building" means a building within which one or more persons reside, work or are served for four or more hours a day; and two or more days a week; and eight or more weeks a year. Without limiting the generality of the foregoing, this includes such developments as school, hospital, food establishment, residences, etc.

** "Working area" means, those areas of a parcel of land that are currently being used or will be used for the processing of wastewater.

5.2.1.4 Seepage Control

The following sections are a summary of the Alberta Environment publication entitled "Design and Construction of Liners for Municipal Wastewater Stabilization Ponds in Alberta". To obtain additional detail, reference should be made to this publication.

1. Control Criteria

The control of seepage from wastewater lagoons is one of the most important aspects of pond design, construction, operation and maintenance. The maximum allowable hydraulic conductivity for pond liners consisting of in-situ material, compacted clay, bentonite and sand, asphalt concrete or other porous materials in which seepage is governed by Darcy's Law, shall be calculated using the following equation:

$$\text{Maximum } K_T = \frac{C \times T}{2 + T}$$

where:

K_T = maximum hydraulic conductivity of liner in the field, being at least one order of magnitude greater than the laboratory value, m/s

T = required or proposed thickness of liner, m

C = 5.2×10^{-9} m/s

2. Site Selection and Investigations

Potential sites for wastewater lagoons shall be critically evaluated in terms of their ability to meet the requirements for seepage control. Preliminary site selection should be based on existing and pertinent topographical, geological, hydrogeological and geotechnical data. The most suitable site(s) based on these considerations shall then be investigated in detail.

The detailed site investigation shall include a review of groundwater use in the area, a reconnaissance of surface features (such as bedrock outcrops and drainage courses) and a subsurface exploration consisting of a minimum of either 5 boreholes or 1 borehole per 2 ha of cell area, whichever is greater. The number of boreholes drilled will depend on the complexity of the geology determined by the initial drilling program. Additional boreholes would be required to delineate a site with features such as sand layers or large sand lenses. Each borehole should be drilled to a depth of 6 m below the proposed invert elevation and at least one borehole should be drilled to a depth of either 20 m or to auger refusal in bedrock, whichever occurs first. At least three boreholes should penetrate the groundwater table in order to determine groundwater conditions including water-table elevations, flow direction and gradient. In situations where a shallow aquifer has been identified as part of the site reconnaissance or drilling programs, nests of wells should be installed to evaluate vertical groundwater flow conditions. Each borehole should be logged and soil samples taken; upon completion of the site investigation each borehole not developed as a monitor well shall also be properly sealed. In-situ and laboratory soil tests as outlined in the Manual for "Design and Construction of Liners for Municipal Wastewater Stabilization Ponds in Alberta" should be conducted.

Monitor wells would be used to measure water levels and, once the levels have stabilized, permeability tests can be conducted to determine the hydraulic conductivity of the material in which the monitor well is completed. The wells should be surveyed with respect to elevation and location in order to determine groundwater elevation and the direction of groundwater flow.

Sites in flood plains, sites located above buried channel aquifers, or sites having either hilly terrain, high bedrock, fissured rock formations or high water tables should be avoided to protect against possible groundwater pollution problems and to prevent adjacent lands from being adversely affected by seepage. Areas of high water table should be avoided due to possible construction /design problems. The bedrock surface should be a minimum of 3.0 m below the invert of any cell.

3. Liner Design

The following design requirements shall be used when designing and constructing waste stabilization pond liners:

(i) Natural in-situ liners shall have a minimum thickness of 0.9 m below the entire bottom, shall be relatively uniform, and shall be completely free of hydrogeologic windows such as sand and silt. Engineered sideslope liners shall be provided if the horizontal hydraulic conductivity of the in-situ liner does not meet the seepage control criterion or if berms are constructed with fill material;

(ii) Compacted Clay Liners shall have a minimum thickness of 0.6 m on the bottom and 1.2 m on the sideslope (measured perpendicular to the slope). The liner should be constructed in 150 to 200 mm lifts and compacted to the required density, within the required moisture content range, to achieve the required seepage control criterion. When determining whether the seepage control criterion is met, the actual hydraulic conductivity of the liner in the field shall be assumed to be at least one order of magnitude greater than laboratory values;

(iii) Bentonite and Sand Admix Liners. Bentonite shall only be considered for lagoon liners where mixing with native sands or silts allows a uniform bentonite and sand/silt admix. Only moderate to high swelling sodium bentonite shall be used. The bentonite application rate required to meet the seepage control criterion should be determined by laboratory permeability tests and then increased by 25% to allow for field conditions. The admix liner shall be at least 100 mm thick after compaction, and any portion of the liner which is susceptible to weathering when exposed shall be covered with suitable soil material;

(iv) Asphalt Liners. The only asphalt liners recommended for use in wastewater lagoons are spray-on bitumen over hydraulic asphalt concrete and spray-on bitumen over soil asphalt. Hydraulic asphalt concrete liners shall be a minimum of 100 mm thick comprising two 50 mm lifts with staggered joints. Soils asphalt liners should be mixed with 150 mm of native sandy soil. In all cases asphalt shall be placed over non-frost susceptible base courses or subgrades. The spray-on bitumen covering the hydraulic asphalt concrete or soil asphalt surface should provide a uniform 2 mm thick membrane; and,

(v) Flexible Polymeric Membrane Liners. The minimum thickness of membranes used to line wastewater lagoons is 5×10^{-3}mm (20 mils). Membranes less than 15×10^{-3}mm (60 mils) thick shall be covered with a 300 mm layer of fine-grained soil on the pond sideslopes to prevent liner damage. PVC and other membrane liner materials that are susceptible to weathering when exposed shall be covered with soil on both the side slopes and bottom. A stable and well prepared subgrade and proper membrane installation (with particular emphasis on seaming) is necessary for successful performance of the liner. A system for venting gas generation beneath the liner should be considered.

4. Groundwater Monitoring

A post-construction groundwater monitoring program is required to assess the performance of the liner.

The monitor wells installed during the detailed site investigation could be used for groundwater monitoring after completion of the ponds and associated facilities. If some or all of the wells have been lost during construction, new monitor wells should be installed where required.

Monitor wells should be constructed of at least schedule 40 PVC pipe that is a minimum of 50 mm in diameter. The screen should be of the same materials and should be machine slotted. Pipe connections should be threaded rather than glued to prevent the introduction of contaminants into the well. A sand or gravel pack must be placed in the annulus around the screened section and should extend not more than half a metre above the screen. The remaining annulus should be sealed to surface with bentonite or a cement slurry. The portion of the pipe sticking up above ground should have a metal protector or other appropriate barrier system installed around it. Wells should be provided with lockable caps to prevent tampering.

Monitor wells shall be located close to the toe of the perimeter berms. The depth of completion will be governed by the information obtained during the site investigation phase. It is important, however, that wells intersect the water table. A minimum of four wells shall be placed around the pond with the well spacing ideally not to exceed 100 metres.

The following groundwater monitoring program shall be instituted for all new wastewater lagoons:

! Wells shall be analyzed for physical, chemical and biological parameters four times in each quarter of the first year of operation. This will provide the baseline data to be compared with future analysis.

The first analysis from each monitoring well shall be undertaken prior to putting a new lagoon into operation. The following three analyses shall be carried out approximately three months apart to cover all seasons in the year.

Water samples shall be analyzed for routine water chemistry, total Kjeldahl nitrogen and chemical oxygen demand.

! Wells shall be monitored for water levels whenever a sample is collected for chemical analysis, and also during the lagoon discharge period. During the lagoon drainage period, one set of readings shall be taken:

i. immediately before discharge,
ii. immediately after discharge, and
iii. approximately one month after the end of the discharge period.

5.2.1.5 Construction Features

There are a number of design and construction features which should be followed in order to facilitate good operation and maintenance of the wastewater lagoons.

1. Berms

(i) Embankment tops shall have a minimum width of 3 m to provide a driving surface for maintenance vehicles;

(ii) Embankment slopes shall be as steep as the safe operation of equipment will permit and the local soil condition will allow. Slopes for (a) the outside of the berm of 4:1 to 5:1 (horizontal to vertical), and (b) the inside of the berm of 3:1 or less, are recommended;

(iii) Inside embankment slopes on the windward side of the prevailing wind shall be armoured with rip-rap or other suitable material. The required size of riprap may be greater where large cells are constructed in high wind areas;

(iv) The "freeboard", the vertical distance between the high water level and the top of the berm, shall be a minimum of 0.6 m to allow for fluctuation of the operating high water level in the cell. Increased 'freeboard' may be required where high winds and steep embankments result in water scouring or to accommodate additional flows in an anomalous precipitation year when evaporation losses are taken into account in sizing the storage cells; and,

(v) Special soils conditions may require the berm(s) to be "keyed" into the subsoil in order to preclude the horizontal seepage across the base of the berm. Determination of whether the "keying in" or "cut-off" procedure is appropriate for berm construction will be based on a geotechnical evaluation and engineering soils report.

2. Inlet Structures

Control manholes are commonly used for access to inlet piping and to regulate the flow from the influent pipeline to the anaerobic or facultative cells. The invert of the influent pipeline entering the control manhole should be above the maximum operating level of the anaerobic or facultative cell. The inlet pipe from the manhole to the anaerobic or facultative cell should enter the cell at approximately 1/3 to 2/3 the cell depth.

In the design of inlet structures, consideration should also be given to receive truck-haul sewage from holding tanks.

3. Outlet and Drain Structure

The drain from the final storage cell shall be installed to ensure that a minimum of 150 mm of liquid is retained. A manhole shall be provided to house a valve and vertical overflow pipe for the drainage pipe, and the drainage valve shall be equipped with a long stem so that it can be operated without entering the manhole. Erosion protection shall also be provided at the location of effluent discharge.

The drain from the final storage cell shall have capacity to ensure that the annual discharge of final effluent is completed in a period of three weeks or less.

4. Flow Measurement

Section 7.2.4.2 (5) outlines the requirements for wastewater flow measurement at pumping stations. In cases where the system is a total gravity system, i.e. no pumping station required to transport flows to the wastewater lagoons, and the system has a capacity greater than 500 m^3/d, a portable or permanent flow measuring device shall be provided at the inlet of the wastewater lagoons.

5.2.2 Wastewater Evaporation Lagoons

Where conventional wastewater lagoons or mechanical plant are impractical because of effluent discharge restrictions, it may be necessary to consider the provision of wastewater evaporation lagoons.

5.2.2.1 System Components and Configuration

For systems with average daily design flows of less than 250 m^3, the system may be designed with one evaporation cell. Provision must be made at the inlet to the cell for settlement and removal of sludge. For systems with flows larger than 250 m^3/d, the evaporation cell shall be preceded by two anaerobic <u>or</u> two facultative cells.

1. Anaerobic Cells

Anaerobic cells shall operate at a minimum depth of 3.0 m and for each cell to retain influent flow for a two-day period based on average daily design flow.

2. Facultative Cells

Facultative cells shall be designed with a maximum operating depth no greater than 1.5 m, and for each cell to retain influent flow for a thirty-day period based on average daily design flow.

3. Evaporation Cells

When establishing the size of an evaporation lagoon, consideration shall be given to local meteorological conditions such as rainfall, evaporation, and evapotranspiration. These factors shall be combined with the design influent flows to calculate the required surface area of the cell(s). In no case shall an evaporation lagoon provide less than 3 years of storage capacity based on average daily design flows. In practice, more than 3 years of storage will be required in all but the most southern areas of Alberta.

Evaporation cells shall have a depth of no greater than 1.5 m.

5.2.2.2 Design Considerations

Design requirements outlined for wastewater lagoons in Sections 5.2.1.3, 5.2.1.4 and 5.2.1.5 also apply to evaporation lagoons.

5.2.3 Mechanical Wastewater Treatment

5.2.3.1 General Requirements

Mechanical wastewater treatment plants shall be designed such that the treated effluent quality meets the performance standards stipulated under Section 3.1 if the effluent is discharged to surface waters, and the standards stipulated under Section 3.2 if the effluent is disposed to land.

5.2.3.2 Design Considerations

1. Setback Distances

Setback distances from mechanical wastewater treatment plants are required to prevent the occurrences of objectionable odours in subdivision when plants are operated normally and within designed capacities. Table 5.4 outlines the minimum horizontal setback distances from mechanical plants including aerated lagoons.

2. Others

Design features for site constraints, fencing, signs and access shall be in accordance with Sections 5.2.1.3(3), (4), (5) and (6) respectively.

5.2.3.3 Safety

Adequate provision shall be made to effectively protect the operator and visitors from hazards. The following shall be provided to fulfil the particular needs of each plant:

1. Enclosure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals;

2. Hand rails and guards around tanks, trenches, pits, stairwells, and other hazardous structures. Height and size of hand rails to conform to Alberta Building Code standards. Materials to be non-corrosive;

3. Gratings over appropriate areas of treatment units where access for maintenance is required. Grating material to be non-corrosive (carbon steel not acceptable);

4. First aid equipment;

5. "No Smoking" signs in hazardous areas;

6. Protective clothing and equipment, such as self-contained breathing apparatus, gas detection equipment, goggles, gloves, hard hats, safety harnesses, etc.;

7. Portable blower and sufficient hose;

8. Portable lighting equipment;

9. Gas detectors;

10. Appropriately placed warning signs for slippery areas, non-potable water fixtures, low head clearance areas, open service manholes, hazardous chemical storage areas, flammable fuel storage areas, etc.;

11. Provision for confined space entry in accordance with Alberta Occupational Health and Safety Act and Regulations.

12. Provision for anaerobic digesters and sludge holding tanks in accordance with the latest edition CAN/CGA - B105, Code for Digester Gas and Landfill Gas Installations.

TABLE 5.4

SETBACK DISTANCES FROM MECHANICAL TREATMENT PLANTS INCLUDING AERATED LAGOONS

Minimum setback distance (m) from the "working area" of the operating mechanical treatment plants or aerated lagoons to:	
The property line of the land where the operating mechanical treatment plants or aerated lagoons is located	30
The designated right-of-way of a rural road or railway	30
The designated right-or-way of a primary or secondary highway	100
An "Occupied Building" where the operating mechanical treatment plants or aerated lagoons serves a designated municipality	300
Any "Occupied Building" on the property of a privately owned rural development which the operating mechanical treatment plants or aerated lagoons serves	100

***** "occupied building" means a building within which one or more persons reside, work or are served for four or more hours a day; and two or more days a week; and eight or more weeks a year. Without limiting the generality of the foregoing, this includes such developments as school, hospital, food establishment, residences, etc.

** "Working area" means, those areas of a parcel of land that are currently being used or will be used for the processing of wastewater.

5.2.3.4 Water Supply and Sanitary Facilities

1. General

For mechanical wastewater treatment plants, an adequate supply of potable water under pressure should be provided for use in the laboratory and for general cleanliness around the plant. No piping or other connections shall exist in any part of the treatment works which, under any conditions, might cause the contamination of a potable water supply.

2. Direct Connections

Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:

(i) Lavatory;
(ii) Water closet;
(iii) Laboratory sink (with vacuum breaker);
(iv) Shower;
(v) Drinking fountain;
(vi) Eye wash fountain; and
(vii) Safety shower.

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

3. Indirect Connections

Where a potable water supply is to be used for any purpose in a plant other than those listed in subsection 2, a break tank, pressure pump, and pressure tank shall be provided. Water shall be discharged to the break tank through an air gap at least 150 mm above the maximum flood line or the spill line of the tank, whichever is higher.

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

4. Separate Non-Potable Water Supply

Where a separate non-potable water supply is to be provided, a break tank will not be necessary, but all system outlets shall be posted with a permanent sign indicating the water is not safe for drinking.

5. Sanitary Facilities

Toilet, shower, lavatory, and locker facilities should be provided in sufficient numbers and convenient locations to serve the expected plant personnel.

5.2.4 Aerated Lagoons

5.2.4.1 General Requirements

Aerated lagoons shall be of the "completely mixed type". The design of completely mixed aerated lagoons requires enough oxygen transfer to satisfy the applied BOD loading and sufficient mixing to maintain a uniform solids concentration in the complete mix cells.

The system shall be designed such that the treated effluent quality meets the performance standards stipulated under Section 3.1 if the effluent is discharged to surface waters, and the standards stipulated under Section 3.2 if the effluent is disposed to land.

5.2.4.2 System Components and Configuration

Typical completely mixed aerated lagoons, treating domestic wastewater, shall consist of the following:

1. A completely mixed cell, having a total retention time of at least two days based on maximum monthly average daily design flow. Aeration equipment in this cell shall be designed to achieve complete mixing by maintaining a uniform solids concentration;

2. A minimum of two partially mixed aerated cells, having a total retention time of at least 28 days based on maximum monthly average daily design flow. The two aerated cells may operate in series or in parallel, with each cell sized to 50% of the maximum monthly average daily design flow; cells in series are preferred for continuous discharge of treated effluent. Aeration equipment in this cell shall be designed to maintain a total dissolved oxygen concentration of at least 2.0 mg/L during peak loading periods; and

3. a polishing cell having a minimum hydraulic retention of five days based on maximum monthly average daily design flow is required for continuous discharge systems.

Operating depths of 5 m are recommended for good mixing efficiency and to reduce heat loss during winter months.

The foregoing configuration and the retention times may be varied if the influent characteristics are considerably different from the typical characteristics of domestic wastewater (BOD - 200 mg/L, TSS - 200 mg/L). Any variation shall be substantiated by the municipality submitting the design calculations for the modified lagoon system.

5.2.4.3 Design Considerations

A number of the design requirements outlined for wastewater stabilization ponds and mechanical wastewater treatment plants also apply to aerated lagoon systems. These design features which shall be applied to aerated lagoons include:

1.	Site constraints	-	5.2.1.3 (2)
2.	Fencing	-	5.2.1.3 (3)
3.	Signs	-	5.2.1.3 (4)
4.	Access	-	5.2.1.3 (5)
5.	Surface Runoff	-	5.2.1.3 (6)
6.	Seepage Control	-	5.2.1.4 (1-4)
7.	Construction Features	-	5.2.1.5 (1-4)
8.	Setback Distances	-	5.2.3.2 (1)

5.2.5 Disinfection

5.2.5.1 General Requirements

Municipalities serving current population greater than 20,000 and discharging to surface waters shall provide disinfection of the effluent to meet the performance standards stipulated in Section 3.1. The need for wastewater disinfection for municipalities serving a current population less than 20,000 with continuous discharge, will be determined on a site specific basis, based on receiving water assessment. Municipalities disposing on land (golf courses, parks, wetlands) shall also provide disinfection of the effluent to meet the performance standards stipulated in section 3.2. In general, where a public health hazard may be created by discharge of wastewater plant effluent, disinfection shall be required.

Ultraviolet irradiation, chlorine or chlorine derivatives are commonly approved and used for wastewater disinfectants in Alberta; alternatives include ozone, bromine, iodine, and gamma radiation. The choice of wastewater disinfectant should be based on a number of factors including flow rates, impact on the receiving stream, disinfectant application and demand rates, wastewater pH, the costs and availability of feed equipment, the costs and availability of specific chemicals, and the operation and maintenance factors.

When chlorine is used as the disinfectant, dechlorination of wastewater effluent is required to reduce the toxicity due to chlorine residuals. End of pipe limit for chlorine will be based on receiving water assessment.

5.2.6 Wastewater Treatment Chemicals

Chemicals selected for use in wastewater treatment plants must be such that they will not adversely affect the operation of the wastewater or sludge treatment processes and will not leave dangerous residuals in the effluent or sludge leaving the plant. The purity of chemicals proposed to be used should be determined. Waste streams from industry, such as ferrous chemicals, can be used provided that they are not contaminated with other hazardous materials.

See section 4.4, "Water Treatment Chemicals" for information on Labels and Material Safety Data Sheets and Storage Handling.

5.2.7 Effluent Disposal

5.2.7.1 Disposal to Surface Waters

1. Seasonal Discharges

The drainage course receiving seasonal discharges from a wastewater treatment plant must be capable of transporting the effluent to the ultimate receiving watercourse without the occurrence of either flooding of adjacent lands or erosion of the drainage course itself.

Wastewater lagoons shall be drained once a year between late spring and fall; the discharge period should not exceed three weeks unless the local conditions preclude this rate of discharge. Early spring discharges may be allowed only under exceptional circumstances to comply with any local conditions.

Where drainage course improvements are required to handle a seasonal pond discharge, prior approval must be obtained under Water Act in Alberta. In addition, the owner of the facility should obtain easements across privately owned land along the drainage course in areas where flooding problems are foreseen.

2. Continuous Discharges

The drainage course receiving discharges from a wastewater treatment plant must be capable of transporting the effluent to the ultimate receiving watercourse without the occurrence of either flooding of adjacent lands or erosion of the drainage course itself.

Piped systems are the preferred method of transporting continuous effluent discharge to an approved receiving watercourse. Under some conditions, however, it may be permissible to use a man-made or intermittent natural drainage course to convey the continuous discharge for a short distance to the receiving watercourse.

5.2.7.2 Disposal to Land

In Alberta, the most common method of land disposal of effluent is through wastewater irrigation, and to a lesser extent through wetlands. Rapid infiltration is generally uncommon, and, so far, has not been practiced by any municipality in Alberta.

1. Wastewater Irrigation

Minimum treatment requirement and treated effluent quality standards for wastewater irrigation are outlined in Section 3.2.1.

Wastewater irrigation, as an effluent disposal option, should be considered only when it is environmentally acceptable and agriculturally beneficial. This may be suitable in regions where additional moisture can be utilized for improved crop production. Application amounts should be based on the net consumptive use of the crop being grown while taking into account the seasonal moisture deficiencies, application efficiencies, and any additional needs as may relate to leaching requirements. The primary objective should be the enhancement of crop production. The root zone of productive soils can also serve as one of the most active media for the decomposition, immobilization, or utilization of municipal wastes. An added benefit of wastewater irrigation is that wastewater may be safely released to the environment at somewhat lower levels of treatment than would apply for other disposal options.

Wastewater suitability for irrigation purposes is based on specific water quality parameters. These parameters should be tested prior to and during their release. Site acceptability is to be based on pertinent soil and geologic properties, topography, hydrology, climate, and zoning and cropping intentions.

Land application of wastewater shall be in accordance with the latest edition of Guidelines for Municipal Wastewater Irrigation, published by AEP.

2. Rapid Infiltration

In the rapid infiltration process, wastewater is applied at high rates for a period from several days to several weeks, and allowed to infiltrate and percolate into the soil. A rest period is then required for the infiltration and treatment capacity of the soil to be restored. The process can be used for wastewater treatment where soil and geologic conditions provide rapid infiltration and high permeability. This process may have a deleterious effect on both groundwater and surrounding surface water.

Design of rapid infiltration (sometimes referred to as infiltration-percolation) systems shall be done in accordance with the joint Alberta Environment - City of Red Deer publication entitled Rapid Infiltration - A Design Manual. In general, the following shall apply:

(a) A minimum of primary treatment;

(b) A minimum of two wastewater storage cells to provide for periodic basin maintenance during the rest period;

(c) Detailed multidisciplinary site investigations that include, but is not limited to soils, hydrology, hydrogeology, topography, and climate;

(d) Subsurface groundwater flow definition;

(e) Subsurface drains as required.

3. Wetlands Disposal

Disposal of effluent on wetlands shall be considered only if the wastewater has received secondary or tertiary treatment and the effluent quality meets the standards specified in Tables 3.1 and 3.2.

Wetlands shall be evaluated and designed in accordance with the Alberta Environmental Protection publication entitled Guidelines for the Approval and Design of Natural and Constructed Treatment Wetlands for Water Quality Improvement.

5.2.8 Biosolids (Sludge) Disposal

Biosolids are principally organic in nature, and contain varying quantities of metals, nutrients, salts, grit, synthetic organics and pathogenic organisms. The exact composition of biosolids is a function of the wastewater being treated and can vary significantly from municipality to municipality depending on the quantity and quality of industrial, commercial and institutional inputs. The quantity, quality and characteristics of biosolids from wastewater treatment may be altered by employing various treatment techniques (see Section 7.4).

In general, as treatment efficiency increases the volume of biosolids increases. Biosolids must be disposed of in an environmentally acceptable manner if wastewater treatment, as a whole, is to be considered effective. Commonly employed biosolids disposal methods include:

1. ocean dumping;
2. sanitary landfilling;
3. incineration;
4. permanent lagoon storage; and
5. land application.

Of these disposal methods, the only way of re-utilizing and conserving the useful constituents in biosolids is through its application to land. This method is also consistent with the objective of disposing of biosolids in an environmentally acceptable manner.

The application of biosolids to agricultural lands can benefit both municipalities and farmers. To municipalities land application is often the most practical method of biosolids disposal and to farmers biosolids is an excellent potential soil conditioner and fertilizer.

Land application of biosolids shall be carried out in accordance with the latest edition of Guidelines for the Application of Municipal Wastewater Sludges to Agricultural Lands published by AEP. The purpose of these guidelines is to:

1. discuss the risks and benefits associated with the use of biosolids on agricultural land; and

2. outline biosolids application rates and procedures that should be used when applying biosolids to agricultural land in Alberta.

In addition to being a potential soil conditioner and fertilizer, biosolids may also contain pathogenic organisms and quantities of elements and chemicals which can adversely affect crop quality and yield and result in pollution of surface and groundwaters. These guidelines represent an attempt to provide criteria which will maximize the benefits associated with land application of biosolids while minimizing the potential risks.

5.2.9 Health and Safety Act

The design and construction of all components of the wastewater system shall conform to the safety provisions of the Alberta Occupational Health and Safety Act and Regulations.

6.0 DESIGN GUIDELINE - WATERWORKS SYSTEM

6.1 Design Criteria

6.1.1 Water Quantity Requirements

A general correlation exists between the available quantity of drinking water and the level of public health within a community. The waterworks system must be capable of providing sufficient quantities of water to meet the needs of consumers, meaning that the proposed source of supply should adequately meet the demand of consumers without any adverse effects on other water users. Water quantity requirements should be based on an assessment of all existing and possible future domestic, institutional, commercial and industrial demands, and should also consider possible water demands for fire fighting purposes.

Water is also a natural resource which should be managed and used in a responsible manner. Water conservation measures that eliminate water misuse or wastage should be implemented; water metering is a particularly effective method of encouraging the responsible use of water. The use of water saving fixtures, upgrading or replacement of leaking water distribution piping, water use restrictions, and proper water treatment plant operation are other water saving measures which should be considered when determining water requirements.

6.1.2 System Capacity

Various components of waterworks systems should have a design life that is compatible with the function of the component. For example, a water treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 or 25 year design flow. Intakes and outfall structures, which have high base construction costs, should be designed for the entire design horizon which is usually about 20 to 25 years. Storage facilities, on the other hand, should be phased to avoid operational problems (such as increased chlorine demand or oversized pumps) which are associated with excess storage and detention times.

6.1.3 Raw Water Supply and Water Treatment

The raw water supply and water treatment plant should be designed for at least 110% of the projected maximum daily design flow. This compensates for accumulated in-plant losses of up to 10% of the produced treated water.

6.1.4 Water Distribution

Water distribution system should be designed to handle a normal operating pressure between 350 kPa and 550 kPa under a condition of maximum hourly design flow. In order to avoid damage to household piping, maximum system pressure should not exceed 700 kPa.

In order to provide adequate service, the minimum distribution pressure during peak demand design flow should be 150 kPa.

In addition to the maximum operating pressures, there are transient pressures due to pump starts and stops, power failures, or rapid valve operation. Pumps should be designed to minimize these surges, and watermains should be designed to withstand these surges, in addition to the maximum operating pressure.

At the discretion of municipalities, distribution mains may be located on the sites of undeveloped road allowances or on the verge of developed roads.

6.2 Raw Water Supply

6.2.1 Water Source/Quality

Raw water from a selected source should be of sufficient quality such that it can be economically treated to produce a finished water which complies with the potable water quality and the performance requirements outlined in Section 2.0. Factors which influence the choice of the raw water source should include reliability, treatability, environmental impact, and economics. The choice of filtration process should be based on total coliform count, turbidity and colour as presented in Table 6.1.

As the level of treatment required would be dependent on the raw water quality, the owners may develop watershed protection programs to reduce any potential risk of source pollution. The owners may maintain a sanitary control area around all sources for the purpose of protecting them from existing and potential sources of contamination. The owners may also develop a watershed control program, identifying land ownership and activities which may adversely affect source water quality and develop watershed control measures, including documentation of ownership and relevant written agreements and monitoring of activities and water quality.

6.2.2 Site Selection Criteria

Several factors which should be considered when selecting a site for new or expanded water supply and treatment works include:

1. isolation from non-compatible land users;

2. facility location with respect to the raw water source, the area(s) being serviced, and proximity to associated utilities;

3. physical site problems including susceptibility to flooding, subsurface geology, or proximity to natural watershed areas; and,

4. adequacy of the site for future expansion.

TABLE 6.1

GENERALIZED CAPABILITY OF FILTRATION SYSTEMS TO ACCOMMODATE RAW WATER QUALITY CONDITIONS

Treatment	General Restrictions		
	Total Coliforms (#/100 mL)	Turbidity (NTU)	Colour (TCU)
Conventional with pre or in-plant disinfection*	< 20,000***	No restrictions	< 75
Conventional without pre or in-plant disinfection*	< 5,000	No restrictions	< 75
Direct filtration**	< 500	< 7-14	< 40
In-line filtration**	< 500	< 7-14	< 10
Slow sand filtration	< 800	< 10	< 5

* Must insure control of disinfection by-products
** When TOC > 3 mg/L turbidity reduction is impaired
*** When total coliforms > 20,000/100 mL, or colour > 75 TCU, additional treatment may be required

Note: Ideally pilot testing should be conducted to demonstrate the efficacy of the treatment alternatives.

6.2.3 Surface Water Supply

6.2.3.1 Intakes

1. Sizing

For the reasons discussed under Section 6.1.2, intakes should be sized for an extended design period (as compared to a phased design), which is usually about 20 to 25 years.

2. Design

Intake design should account for wave action and should provide adequate protection against the effects of ice and boat anchors. Intakes should be identified with buoys or reflectors where in proximity to shipping or recreational activities. The designer should be familiar with the requirements as legislated under the Navigable Waters Act.

The inlet should be located to prevent bottom sediments from being picked up. For small intakes, consideration should be given to providing means for back-flushing the intake, if practical.

The design of river intakes differs from that for lakes and stagnant water bodies in that more secure anchoring is required to resist bottom scouring and stream velocities. River intakes should be equipped with trash racks and should also be located well upstream from potential sources of pollution.

An acceptable alternative design to direct intake is an infiltration gallery intake. This type of intake is suitable when the river bed is composed of gravels and rocks or if the floodplain is demonstrated to have a high water table that is connected to the nearby watercourse. Items to be considered are:

(i) the sediment load in the river (may necessitate backwashing or aeration provisions);

(ii) the use of filter cloth; and

(iii) the depth of perforated infiltration pipes (to be located as deep as possible in the aquifer so as not to be affected by seasonal fluctuations).

6.2.3.2 Screening

1. Sizing

Screen mesh size should be governed by the surface raw water quality <u>and</u> the species of fish present in the raw water supply. Screen size requirements shall be in accordance with requirements of the Fish and Wildlife Services of the Department.

2. Location and Type of Screen

Screens may be constructed either at the intake structure itself or in-plant just prior to the raw water pumping facilities. For small treatment plants with in-plant screens, two (2) fixed screens in series will suffice, while for larger plants the use of at least two (2) mechanically cleaned screens operating in parallel is recommended. A combination of fixed and mechanically cleaned screens may be used for medium capacity plants.

3. Washing

Fixed screens should have lifting lugs for removal and washing. Screen waste should not be returned to a raw water storage area.

6.2.3.3 Pumping

Pumps should be specified so that the full range of flows anticipated can be provided with pumps operating in the vicinity of their optimum efficiency points, with due regard to the hydraulic design of the discharge piping. This is often accomplished by selecting pumps which have wide band efficiencies and a relatively flat operating curve.

The number of pumps should be consistent with the pattern of flow required and the method of flow control. It is recommended that at least three pumps be provided for operating flexibility; a minimum of two pumps are required, one as standby. Pump capacities should be such that with the largest unit out of service, the remainder will be able to supply the treatment plant capacity as stated in section 6.1.3.

Provision should be made for an individual pressure gauge on each pump and an isolating valve and check valve on the discharge side. Dry well pumps should be provided with suction side valves. The use of slow opening pump discharge should be considered at raw water pumping stations remote from the treatment plant. Piping should be arranged to allow ready disassembly from pump to shut off valves, and include a flexible type coupling to permit proper alignment of the piping and pump. Couplings must be adequately protected against thrust. Pump elbows should be supported to remove all bending moments, either steady or shock, from pump nozzles.

The station design should allow for future additional pumping units and where possible, the pipework should be large enough for an increase in pump size to be accommodated. Adequate space should be provided for the installation of these additional units, and to allow safe servicing of all equipment.

Adequate space should be provided for removal of pumps, and in the case of vertical turbine pumps it may be necessary to provide a roof access for removing the units and sectional discharge pipes so that they can be completely removed from the raw water well.

All piping should be arranged so that there is sufficient room to service all valves and other parts, and to permit their removal with minimum disturbance to the system. A bridge crane, monorail, lifting hooks, hoist or other adequate facilities should be provided for servicing or removing equipment.

Pumps should be mounted on bases above the floor level, and all access openings into the well shall have suitable curbs around them to prevent floor drainage entering the well. The station floor should be sloped to floor drains. Floor drainage back to the raw water well is not permitted. Drainage from pumps on to the floor is not acceptable.

The pumps should be capable of supplying the water over the entire range of flows to be treated. This could be achieved through the provision of pumps with variable speed motors or through control valves. Where this is accomplished by control valves, it is normal practice to use butterfly valves operating in a range to maintain stable control and avoid cavitation. At small treatment plants where substantial seasonal variations in flow exist, it may be necessary to provide duplicate flow control systems - one suitable for very low flows (which normally occur in winter) and one suitable for the plant design flow.

6.2.3.4 Raw Water Storage

Raw water storage improves water quality by providing presedimentation of solids, ensures an adequate supply when a stream or lake source is intermittent, and provides standby against failure of intake facilities. It also enables the operator to avoid the undesirable practice of drawing water during periods of poor raw water quality, allowing a low rate of withdrawal at the source.

1. Facility Planning

The designer should assess the need, location, and sizing of the raw water storage reservoir before proceeding with final design. Reservoir sizing should be determined by assessing the availability of water and the nature of upstream activities. The designer should also consider any potential adverse effects on the water intake, storage, or treatment facilities; and should include design features to minimize the effects of fluctuating raw water turbidity.

2. Multi-Cell Provision

Raw water reservoirs should be constructed with a minimum of two cells. This will enable the plant operator to withdraw raw water from the second cell when the first cell is being filled or repaired. Each cell should be sized to retain about 75% of the annual raw water needs. In areas of drought, the number and storage capacity of each cell should be increased to overcome long term droughts.

Control structures should enable the plant operator to isolate each cell, to drain each cell, and to enable the cells to be operated in series or in parallel. A bypass around the reservoirs may also be provided to obtain water during those periods when reservoirs are out of service.

Each cell should be deep enough to restrict light penetration within the depth of the reservoir to discourage the development of ideal habitats for aquatic plants.

Inside slopes of the cells should be armoured, where required, to prevent erosion. The impact of ice formation on winter storage should be accounted for in the design.

3. Reservoir Management

The owners of raw water reservoirs should have a reservoir management program that identifies the current condition of the reservoir, the necessary storage capacity, and the necessary management procedures to respond to changes in reservoir conditions.

The reservoirs should be managed to avoid any difficulties with taste, odour, colour, iron and manganese in drinking water. In-reservoir management techniques should address problems with algae, weeds, low dissolved oxygen, and loss of storage capacity.

Artificial circulation, aeration, algicides, phosphorus precipitation, sediment removal, dilution, and flushing are reservoir management techniques that should be adopted to improve the water quality.

4. Lining

Raw water reservoirs should be designed to minimize seepage. Reservoirs should be lined in accordance with Section 5.2.1.4 pertaining to wastewater stabilization pond liners. This standard is based upon AEP's publication entitled <u>Design and Construction of Liners for Municipal Wastewater Stabilization Ponds</u>.

6.2.4 Groundwater Supply

6.2.4.1 Siting of Wells

Wells should be located to avoid proximity to sources of pollution and/or flooding. Wells shall be at least 100 m upgradient from pollution sources such as septic tanks, drainage fields, cesspools, or wastewater stabilization ponds; wells should not be located near sanitary landfill sites, underground fuel storage tanks, or cemeteries. Reasonable access shall be provided for repair and maintenance.

6.2.4.2 Well Protection

In order to protect the finished supply structure from external contamination, the following should be provided:

1. Water-tight construction to at least 6 m below ground level. This depth may be increased if local conditions present a danger of surface contamination;

2. An annular opening of at least 40 mm outside the protective casing, filled with an approved grouting material; and,

3. Other precautions in the design to seal off undesirable subsurface formations and surface contamination.

6.2.4.3 Pumphouse Design

The design criteria for well pumping stations generally follow those presented for raw surface water pumping, and standby pumping facilities should be provided which are capable of maintaining normal servicing standards. In addition, the following special considerations apply:

1. The elevation of the top of the production well casing should be 200 mm above the established ground level or the pumphouse pit floor, and at least 600 mm above the highest recorded flood level;

2. A pump pedestal should be provided around the surface casing to support the full weight of the pump and to prevent any weight from being placed on the production casing or any associated well casing;

3. A water-tight seal should be provided between the pump base plate (or submersible discharge head) and the pump pedestal, and between the well casing and the pump discharge column to prevent the entrance of contaminants;

4. An aperture for air venting with proper screening should be provided to the production well surface casing. Where there are indications of excessive quantities of explosive or toxic gases in the water, both the well casing and pump columns should be vented to the outside of the pumphouse (protection against freezing is required);

5. Return pipes that will permit water to be recirculated down the well should be avoided as they may cause contamination of the well. In cases where recirculation is proposed because of severe water shortages, the proponent should provide design details with the application for a permit;

6. The well should not be located within 1.2 m of an exterior wall of the pumphouse, and should be centred under a hatchway in the roof which is at least 1 (one) metre square to facilitate access. Also, to accommodate redevelopment of wells, access for service rigs should be provided;

7. Well water quality monitoring should be provided by including a suitable sampling point. Water level monitoring should be provided by including at least one opening in the well head which allows vertical access to the inner casing for equipment installation;

8. Either an electric resistance tape or a water level measuring airline should be installed (clamped to the pump column) complete with a suitably calibrated pressure gauge;

9. The piping layout in the pumphouse should include an in-line free discharge pipe to the outside of the building to permit future testing of the well; and,

10. A flow measurement device should be provided.

6.2.4.4 Well Disinfection

Prior to the use of a water well for domestic consumption, the well should be disinfected. Chlorine should be applied to ensure that a concentration of 50 mg/L is present in the well for a period of twelve hours. Dosage should be computed on the basis of water required to provide mixing throughout the entire well volume.

6.3 Water Treatment

6.3.1 Rapid Sand Filtration

The following guidelines have been prepared to document the desirable ranges, and the normal, minimum or maximum acceptable levels for the various design parameters used in the design of rapid sand filtration systems. This document is not a design manual per se as documentation of all parameters relating to water treatment plant design is beyond the scope of these guidelines, but an attempt has been made to include the parameters of greatest importance from the process and reliability standpoints as a guide.

6.3.1.1 General Guideline

1. Process Selection

(i) Raw Water Characteristics

The raw water quality is the single most important factor in determining the type and the extent of treatment required for a particular source of water. Thus, a thorough evaluation of the raw water types should precede the selection of a treatment process. The major raw water characteristics are microbiological quality, turbidity, pH, alkalinity, colour, TOC, TSS, iron, manganese, algal counts, and temperature.

Preferably a five year history that characterizes the main raw water types should be collected. While this is possible for locations where a water treatment plant already exists, it could be impractical for new locations. Therefore, data that characterizes the main water types for at least one year should be collected as a minimum. Facilities that are located upstream and/or downstream from a proposed site may provide valuable information on the raw water characteristics.

(ii) Enhanced Coagulation

It is recommended that treatment plants using surface waters/GWIs should practice enhanced coagulation for disinfection byproducts (DBP) precursor removal. TOC content of the source water is a good predictor of DBP formation potential, and enhanced coagulation is based on the organic content and driven by the TOC levels. Pre-chlorination should not be practiced at high TOC levels.

Enhanced coagulation is defined by achieving the removal percentages prior to the point of continuous disinfection given the source water alkalinity and TOC concentrations as shown on Table 6.2.

TABLE 6.2

TOC REMOVAL PERCENTAGE

Influent TOC mg/L	Influent Alkalinity, mg/L		
	0-60	60-120	> 120
0-2	No action	No action	No action
2-4	40%	30%	20%
4-8	45%	35%	25%
> 8	50%	40%	30%

(iii) Coagulant Residuals

Aluminum levels in finished water has become a concern for systems using aluminum based coagulations, particularly for systems producing water with elevated pH. The owners should be aware that aluminum is under review for possible addition to the GCDWQ. Thus, municipalities should take steps to reduce the amount of aluminum in the finished water, especially for those plants practicing lime softening.

(iv) Jar Tests

As bench-scale testing of the treatment often gives meaningful insight to full-scale results, it is recommended that jar tests be done on the raw water. Jar test data should statistically establish the treatment requirements of the source water with respect to the process and the water treatment chemicals that are being proposed. The recommended apparatus are a paddle type jar tester, 2.0 litre square beakers, automatic pipettes and a temperature controlled water bath. A description of the techniques used, the jar tests results and the conclusions made based on the jar tests should be included in the pre-design report.

(v) Pilot Tests

Where practical, pilot studies should be conducted, and specifically when a non-conventional treatment process is proposed. Piloting should be done for sufficient length of time to statistically verify the proposed treatment process. A description of the apparatus, the results and the conclusions made based on the pilot studies should be included in the pre-design report.

2. Optimization Capability

Unit processes should be designed in order to provide the operator with the flexibility to optimize and integrate each unit. Package plant installations should not be excluded from the requirement to provide the same optimization capabilities as larger plants.

By-passes should be provided for each process unit and the piping sized for the design capacity of the plant.

Manual or automatic sampling capabilities should be provided at each stage of the process. Multiple sampling locations should be provided for a unit such as a clarifier or flocculator.

Prior to commissioning, a tracer study should be conducted in order to prepare a blueprint of the plant hydraulics that can be used for plant optimization and for troubleshooting future problems. A tracer study should be performed at the initial and at the design flow rate and should quantify actual detention as opposed to theoretical detention times for each operating unit. As an alternative, tracer data supplied by the manufacturer for package plants and proprietary systems would be acceptable. The data from the tracer studies should be included in the as-built plans and/or the operating manual.

For plants with throughput in excess of 10 MLD, an actual hydraulic grade line at initial and design flow rates should be measured. This data should be submitted with the design hydraulic grade line as part of the as-built plans and/or the operating manual.

3. Plant Automation

Computer control systems should be provided for all water treatment plants. These will enable the operator to monitor and/or change critical plant operating parameters by means of a computer or a programmable logic controller. Section 6.3.4 identifies those parameters for which monitoring may be automated.

6.3.1.2 Chemical Mixing and Coagulation

Chemical mixing is often the first and also an important step in the process train. Mixing is critical for uniform dispersion of the coagulant with the raw water in order to avoid over or under treatment of the water. An understanding of water chemistry and the process of coagulation - flocculation is extremely important in the design of the components of a rapid-mix unit. The water quality, mode of destabilization and the type of coagulant all play a part in the selection and design of the appropriate unit.

Table 6.3 provides some basic information on the selection of treatment methods and primary coagulants for different raw water quality. The designer is well advised to verify this by undertaking bench-scale or pilot tests. Scale-up, based on geometrically similar units with constant $N^{5/8}D^{5/9}$ for flocculation is recommended; where N = rotational speed (rpm) and D = mixer diameter (m).

Some general comments pertinent for the design of chemical mixing and design criteria for various types of rapid-mix units are as follows:

1. General Comments

(i) The designer should establish the initial and ultimate design flow capacities for each of the different raw water types. The mixers that are selected should be capable of delivering the required energy input for each of these flow rates and raw water types. Unless it can be demonstrated that raw water quality and plant throughput will not significantly change, the rapid mixer should be capable of delivering a range of energy inputs.

(ii) Simultaneous addition of a primary coagulant and a flocculant aid to a single rapid mixer is not recommended. An optimum time of separation should be derived from jar or pilot tests or should be at a minimum of 2 minutes. It is not unusual to add flocculant aids at the flocculator.

Flocculant aids should be mixed into the process stream either mechanically or hydraulically and a maximum energy of mixing should be determined so as not to shear the polymer chain.

(iii) If alum or ferric salts are used, dilution with treated water is recommended in order to enhance mixing. To avoid precipitation of hydroxide forms, alum should not be diluted to below 0.5% (pH of solution is approximately 3.0) and ferric salts should not be diluted to below 2.5%. (pH of solution is approximately 2.0).

The nozzle velocity of a chemical injector into the rapid mixer should not exceed 3.0 m/s since this will result in non-uniform mixing.

(iv) In-line mechanical mixer should be the option of choice for primary coagulants due to its versatility.

Mixing systems consisting of a backmix rector or a channel with one or more mechanical flashmixers are not recommended for primary coagulants since they usually result in a lack of instantaneous mixing, short circuiting, and a mixing time that is excessive for metal salts. These systems are acceptable for chemicals other than primary coagulants.

(v) Injection of chemicals into a pump suction is not recommended since this results in inadequate control of mixing energy and possible pump damage. Hydraulic rapid mixing of a primary coagulant through venturi meters, parshall flumes, weirs and orifices are also not recommended for primary coagulants since these devices provide inadequate control of mixing energy into the process. They are acceptable for chemicals other than primary coagulants.

(vi) High energy mixing is not required for chemicals that are not used for coagulation. As a guideline these chemicals can be injected a minimum of 30 pipe diameters or channel widths away from the point of coagulant addition.

(vii) The materials used in construction of the chemical mixing system should be corrosion resistant.

2. Design Criteria

(i) Rapid Mixing for Inorganic Salts

(a) In-line mechanical mixer

! For Charge Neutralization,
G = 3000 to 5000 s^{-1}
t = 0.5 to 1.0 s
Gt = 2000 to 3000

! For Sweep Coagulation,
G = 700 to 1500 and
3000 to 5000 2 s^{-1}
t = 1 to 7 s
Gt = 5000 to 25,000

(b) In-line Static Mixer and Pressured Water Jets

! For Charge Neutralization,
G = 700 to 1500 s^{-1}
t = 0.5 to 1.0 s
Gt = 500 to 1500

! For Sweep Coagulation,

G	=	700 to 1500 s^{-1}
t	=	1 to 7 s
Gt	=	1000 to 10,000

Note: In-line static mixers and pressure water jets, due to their low energy levels, are not suitable for charge neutralization.

(c) Backmix Reactor (Sweep Coagulation)

G	=	700 to 1000 s^{-1}
t	=	10 to 30 s
Gt	=	10,000 to 25,000

(ii) Rapid Mixing for Polymers

(a) In-line Blender/Backmix Reactor (Charge Neutralization)

G	=	400 to 800 s^{-1}
t	=	30 to 60 s
Gt	=	15,000 to 30,000

(iii) Scale-up Criteria

Constant value = $N^{5/8} D^{5/9}$

Where:

N	=	rotational speed, rpm
D	=	mixer diameter, m

TABLE 6.3

TREATMENT METHODS

Water Quality	Primary Mode of Destabilization	Primary Coagulant	Flocculant Aid	Treatment Method	Recommended Rapid Mixer
High Inorganic Turbidity, High or Low Alkalinity	Sweep Coagulation	Inorganic Salt	If required	Conventional	ILM
High TOC, High or Low Alkalinity	Charge Neutralization	Inorganic Salt	Yes	Conventional	ILM
Low Inorganic Turbidity, High or Low Alkalinity	Sweep Coagulation	Inorganic Salt	Yes	Conventional	ILM, S, BM, PJ**
	Charge Neutralization	Polymer, inorganic salt	No	Direct Filtration	ILM, S*, PJ*, BM*
Low TOC, High or Low Alkalinity	Sweep Coagulation	Inorganic Salt	Yes	Conventional	ILM, S, PJ**, BM
	Charge Neutralization	Polymer, Inorganic Salt	No	Direct Filtration	ILM, S*, PJ*, BM*

*S, PJ and BM may be used with polymers only
**PJ may be used for waters with low alkalinity

Typical Ranges

High Inorganic (Turbidity) > 100 NTU
Low Inorganic (Turbidity) < 10 NTU

High TOC (Colour) > 5 mg/L
Low TOC (Colour) < 2 mg/L

High Alkalinity > 100 mg/L as CaCO3
Low Alkalinity < 30 mg/L as CaCO3

ILM = In-line Mechanical Mixer
S = In-line Static Mixer
PJ = Pressured Water Jets
BM = Backmix Reactor

Notes:

1. For sweep coagulation, waters with low alkalinity should be buffered if inorganic salts that would lower the pH are used.

2. Charge neutralization by inorganic salt, compared to sweep coagulation, would require lower dosage of the salt.

3. A combination of inorganic salts and polymers could be used for optimizing the process, and often provides the best results.

4. Chemicals used as flocculant aid should be added after the addition of primary coagulant, prior to or at the flocculation unit.

5. Colour is best removed in the pH range of 4 to 5.5 with inorganic salts (alum) by charge neutralization; this pH range may not be the optimum for turbidity removal.

6.3.1.3 Flocculation

Some general comments pertinent for the design of flocculation system and design criteria for various types of flocculators are as follows:

1. General Comments

(i) All systems should employ a tapered velocity gradient design.

(ii) All systems should employ proper compartmentalization so as to avoid short circuiting.

(iii) Except in very small plants a minimum of two flocculation trains and three stages should be provided.

(iv) For conventional treatment the flocculation units should be located as close as possible or adjacent to the solids separation units.

(v) The inlet and outlet design should prevent short circuiting and the destruction of floc.

(vi) A drain or pumps should be provided in order to handle sludge removal.

(vii) For mechanical flocculators an infinitely variable speed mixer is recommended for the last stage. Two or three speed motors are acceptable for the preceding stages.

(viii) Baffles may be used in order to improve the efficiency of mixing. Two or four baffles are recommended and they should penetrate 1/8 to 1/12 of the width across the individual mixer compartment.

(ix) Flocculation tanks should be no deeper than 5 meters. Greater depths tend to create unstable flow patterns.

(x) Alternative and proprietary designs will be judged on their own merit. Prior to design, pilot testing of the alternative or proprietary systems should be conducted and the results should be included in the pre-design document. As indicated earlier, scale-up based on geometrically similar units with constant $N^{5/8}D^{5/9}$ is recommended.

(xi) If there is a significant pH depression, the materials that are used for construction should be corrosion resistant. Alternatively, corrosion control measures (e.g. cathodic protection, coatings) could be used.

(xii) Diffused air systems are not recommended due to the high rate of energy consumption and inefficiency. An exception would be if volatile inorganic or organic compounds need to be removed.

(xiii) Water jet mixing is not recommended due to the high shearing force of the jet that tends to restrict floc size.

2. Design Criteria

(i) Vertical Turbine Flocculators

G = 100 to 10 s^{-1} (tapered)
t = 15 to 40 minutes (for conventional treatment)
G x t = 20,000 to 200,000
Stages = 2 (for direct filtration applications) to 4, typically 3
Maximum tip speed = 2.0 m/s
Blade Area/Tank Area = 0.1% to 0.2%
D/T = 0.2 to 0.4 (D = blade length, T = equivalent tank diameter)
Shaft speed = 8 to 25 RPM

! Recommended for direct filtration (higher energy/shorter detention time) and conventional systems (lower energy/longer detention times).

(ii) Horizontal Paddle Flocculators

G = 50 to 10 s^{-1} (tapered)
t = 30 to 40 minutes
G x t = 20,000 to 110,000
Floc stages = 3 to 6
Maximum tip speed = 1.0 m/s
Blade Area/Tank Area = 5% to 25%
Shaft speed = 1 to 5 RPM
Minimum number of paddles per shaft = 3

! Recommended for conventional treatment systems.

Note: Baffled Walls for Mechanical Mixers

Typical orifices size = 100 mm to 150 mm, rectangular or round, evenly distributed.
Maximum velocity of water through the orifices in the first stage 0.60 m/s
Maximum velocity of water through the orifices in the last stage = 0.35 m/s.
Maximum velocity of water through orifices that connect directly into a sedimentation tank = 0.25 m/s.
Headloss through the ports should be less than 1 cm in order to prevent floc break up.
At the bottom of the baffle, clearance should be provided for the washing and removal of sludge.
The top of the baffle should be slightly submerged in order to promote the passage of scum.
Baffled walls contribute a G of 5 to 25 s^{-1} and this may be incorporated into the overall mixing requirement.

(iii) Baffled Channel Flocculators or Hydraulic Flocculators

Commonly called "around-the-end" or "over and under"
"G" = 12.7 $(h/t)^{0.5}$ at 4°, where t = residence time in seconds, and h = headloss in metre

G = 50 to 5 s^{-1} (tapered)
t = 20 to 45 minutes
Maximum flow velocity = 1.0 m/s, downstream sections will have much lower velocities.
Typical headloss = 0.6 to 1.0 m
Minimum water depth = 3.3 m
Minimum distance between baffles = 0.75 m
Minimum number of channels = 6
Typical headloss across the tank = 0.3 to 0.6 m

! Recommended for conventional treatment where plant flow variation is small.

! Adjustable baffle walls are recommended for varying raw water conditions.

6.3.1.4 Solids Separation

1. Sedimentation

Some general comments pertinent for the design of sedimentation/clarification system and design criteria for various types are as follows:

(i) General Comments

(a) The minimum number of sedimentation tanks should be two in separate trains with dedicated filters except in very small plants.

(b) Bottom slope can vary from <1% to 8% depending on the sludge removal method.

(c) Sludge should be automatically removed by mechanical means. Steep sided hopper shaped bottoms (e.g. >45°) by themselves are inadequate for sludge removal unless the sludge bed is not allowed to compact.

(d) Installation of a grit chamber at the low lift pumps is recommended if raw water contains a large amount of silt and sand.

(e) If there is a significant pH depression, the materials that are used for construction should be corrosion resistant. Alternatively corrosion control measures (e.g. cathodic protection, coatings) are recommended.

(f) An overflow to waste should be provided.

(g) A by-pass for the sedimentation unit may be provided to permit direct filtration during maintenance of the tank or when water conditions are suitable for direct filtration. Disinfection should be capable of providing 1.0 log reduction of Giardia during the direct filtration mode.

(ii) Design Criteria

(a) Horizontal-flow type (Rectangular basin without high rate settling modules)

Surface Loading Rate = 0.83 to 2.5 m/h
Water Depth = 3 to 5 m
Detention Time = 1.0 to 3 hours
Length/Width = Minimum 4/1, Recommended 5/1
Maximum Length = 75 m
Maximum Width = 25 m
Freeboard = 0.6 m (typical)
Weir Loading < 11 m^3/m.h.
Reynolds Number <2000
Froude Number >10^{-5}

Due to the potential for short circuiting and dead spaces, rectangular basins should not be designed with 180^o horizontal bends along the flow path.

(b) Horizontal-flow type (Rectangular basin with high rate settling modules)

Surface Loading Rate (Flow/Projected area of basin covered by modules)
= 5.0 to 6.3 m/h (cold water: < 10^{o}C)
= 7.5 to 8.8 m/h (warm water: > 10^{o}C)

Water Depth = 3.6 m (minimum due to mechanical sludge removal) to 5.0 m.
Maximum Area Covered by Tube Settlers = 75%, from the back wall forward to the inlet.
Weir (Launderer) Loading <15 m^3/m.h.
Approach velocity to modules = 0.6 m/min (typical)

! Higher surface loading rates may be acceptable when a heavy floc is generated. Natural conditions or the use of a flocculant aid can produce rapidly settling floc.

! A continuous sludge removal system is recommended if high rate settlers are employed.

! Due to the potential for short circuiting and dead spaces, rectangular basins should not be designed with 180^o horizontal bends along the flow path.

Note: 1. For perforated Inlet baffles in rectangular basins that are not part of the flocculator

- ! Ports should be uniformly distributed across the entire cross section.

- ! While maintaining the structural integrity of the baffle wall, a maximum number of ports should be provided in order to minimize the velocity of the jets and the dead zones between ports.

- ! Maximum velocity of the water jet entering the sedimentation tank should be 0.25 m/s.

2. For long rectangular tanks, provision should be made for the design and installation of an intermediate diffuser wall(s). These walls may be installed at or after plant start-up.

3. For Outlets for Rectangular Basins

- ! Long finger launderers with or without adjustable weir plates are recommended.

- ! Submerged orifice pipes and troughs are acceptable.

- ! Significantly higher weir loading rates may be allowed if the weirs are evenly distributed over a substantial portion of the surface (60% to 70%).

(c) Up-flow Type (Radial)

Circular or square in shape
Surface loading rate = 1.3 to 1.9 m/h
Water Depth = 3 to 5 m
Detention time = 1 to 3 hours
Weir Loading < 7 m^3/m.h.

- ! These are usually proprietary designs.

- ! Recommended for plants where raw water flow and quality is constant.

(d) Reactor Clarifier

Flocculation time = 20 minutes, and up to 40 minutes in cold water
Surface loading rate < 3 m/h for alum/ferric coagulants
Detention time = 1 to 2 hours
Weir Loading < 15 m^3/m.h.

- ! These are usually proprietary designs.

- ! Tube settlers may be added in order to improve performance as plant throughput increases.

- ! Recommended for water softening or conventional treatment where the raw water quality is constant.

(e) Solids Contact Clarifiers (with high rate settling modules)

Flocculation time = 20 minutes typically and up to 40 minutes in cold water
Surface loading rate < 6 m/h for alum and < 9 m/h for lime
Detention time = 1 to 2 hours
Weir Loading = 7.5 to 15 m^3/m.h.
Slurry recirculation rate = 3 to 10 times the raw water inflow rate

- ! Lower values for recirculation are used for coagulation.
- ! Higher values for recirculation are used for lime/soda ash softening.
- ! These are usually proprietary designs.
- ! If high rate settling modules are not used, the loading rates would be about a third of what is recommended.
- ! Recommended for water softening or conventional treatment.

2. Dissolved Air Flotation (DAF)

(i) General Comments

(a) The minimum number of DAF units should be two.

(b) DAF is a particularly useful process where the floc is light (such as often occurs with low-turbidity raw waters) or where the suspended particles (such as algae) tend to float rather than sink.

(c) Hydraulic loading rates are typically much higher than for gravity sedimentation, and there is little advantage to water depths greater than about 2 m.

(d) DAF is not as sensitive to upsets caused by start/stop operation, and a good quality effluent can usually be obtained within a few minutes of start-up.

(e) Uniform withdrawal of the clarified water, from the bottom area of the units, required to minimize short circuiting.

(f) DAF should be considered only in conjunction with chemical pre-treatment.

(g) Suitability of DAF for a particular raw water source should be substantiated/verified by a pilot study.

(ii) Design Criteria

(a) Bubble diameter = 20 to 100 microns

This is typically achieved by air saturation in a pressurized vessel; larger bubbles will shear rather than float the floc particles.

(b) Surface loading rate = 5 to 15 m/h

Dependent on factors such as water quality and temperature, tank configuration, and size and characteristics of suspended particles.

(c) Saturation pressure = 450 to 725 kPa
Recycle rates = 6 to 12% of the total process flow
Air requirements = 8 to 10 grams/m^3 of raw water

Design of the recycle water injection nozzles must ensure even distribution across the inlet, and minimize tendencies for the air bubbles coalesce near the point of injection.

(d) Inlet baffle should be placed at an angle greater than 45^o to the horizontal (typically 60^o to 75^o).

Cross flow velocities between the top of the weir and the water surface = 0.7 to 1.0 m/min.

(e) Floating sludge removal system - Mechanical removal by reciprocating scraper.
Hydraulic removal will result in dilution of the waste sludge.

6.3.1.5 Filtration

1. Filtration Systems

There are basically three types of rapid sand filtration systems. They are: declining rate filtration; influent flow splitting; and constant rate filtration.

(i) Declining Rate Filtration

In declining rate filtration, filter influent enters through specially designed manifolds to provide equal distribution to all filters. The filter outlet contains a restriction limiting filtration rate through a clean bed to the maximum allowable rate, and filtration rate declines as the bed plugs. An advantage of this method is that it avoids potential water quality deterioration caused by high shearing forces on a dirty bed which can occur in constant rate filtration as design headloss is approached. Disadvantages include the need for special care in start-up of filters by gradually opening of the effluent valve, and a loss of operating flexibility which is particularly significant in larger plants.

(ii) Influent Flow Splitting

Influent flow splitting employs free fall weirs as a means of providing equal flow to multiple filters, thus avoiding the need for, and cost of, effluent control systems. The disadvantages include the potential for flocculated particles to shear passing the weir, thus impairing the filtration process; and that weir settings are normally made to accommodate a single plant flow rate. Changes in plant flow cause different losses in influent channels and the influent flow is no longer equally divided.

(iii) Constant Rate Filtration

The most commonly used method is known as constant rate filtration. In this method the filter effluent piping contains a flow measuring device, typically a venturi flow meter, and an automatically controlled modulating butterfly valve. The filtration rate is set to a pre-determined value by positioning the effluent valve accordingly and filtration is continued at that rate by the effluent valve gradually opening to compensate for increased headlosses as the filter bed plugs. The rate is maintained until clearwell level exceeds a level typically 300 mm below the design top water level. At this point the filtration rate decreases proportionally to the 'freeboard' in the clearwell until the filter shuts down when the clearwell is full.

2. General Design Consideration

The design of gravity filters should provide:

- adequate headroom above the filter to permit inspection and operation and maintenance, and provide reasonable access to the filters for observation (e.g., a walkway along the length and width of the filters for package plants);
- an overflow sized for the filter/plant capacity to prevent flooding, unless provided elsewhere in the raw water supply system;
- means of cleaning influent pipes or conduits where solids loadings are high;
- effluent piping designed hydraulically for flows of up to 50% in excess of filtration design capacity to accommodate potential peak demands;
- effluent piping arranged to prevent backflow of air into the filter;
- operation with a minimum water depth in excess of the design terminal headloss to prevent negative pressure and air binding of the filter as described in subsection 5 - Headloss;
- an acceptable method of regulating flow as described in subsection 1 - Filtration Systems;
- controls as described in Section 6.3.4 - Controls and Instrumentation.

Wash water troughs should be designed so that a clearance is maintained between the bottom of the trough and the level of the expanded media during backwashing.

The bottom of trough to the top level of the static media should not be less than 600 mm, and in some cases should be higher, and the trough capacity should be such that the maximum wash water rate can be accommodated with at least 50 mm freeboard. Trough spacing should be such that equal filter areas are served by each trough, and a maximum horizontal travel for suspended particles to reach a trough of 1.0 m is recommended. Troughs should be located so that they do not obscure filters or affect accessibility.

3. Filter Media

The quality of filtered water is a function of both media size and media depth. The selection of filter media determines not only the filter effluent quality but also the filter backwash regime, and thus the backwash requirements become an integral part of the media decision. For instance, the greater the uniformity coefficient of the media, the larger the backwash rate required to fluidize the coarser grains thus provided.

Dual-media filters have some advantages over single-media filters in terms of filter run lengths, headloss, hydraulic loading rate, etc. For dual-media filters, the size of the sand layer must be selected to be compatible with the anthracite that has been selected. The bottom sand layer should have approximately the same or somewhat higher flow rate for fluidization than the anthracite to ensure that the entire bed fluidizes at the selected backwash rate. Further, the effective sizes of the sand and anthracite should be selected to achieve the goal of coarse-to-fine filtration without causing excessive media mixing.

The particle size and the depth of the media selected for a given filter application depends on the kind of suspended solids to be removed by the filter. This is best established by pilot studies. Typical design data is shown in Table 6.4.

Sieve analyses should be performed and the values plotted to ascertain sand size distribution.

Deep bed filtration, i.e. > 2.0 m, may be used in conjunction with the policies and procedures outlined in Section 5.2 - Unproven or Innovative/Alternative Technologies, of the Guiding Principles/ Policy Document (1996).

4. Filtration Rates

Filter loading rate is an important parameter in the design of a water treatment plant; and in general the filters may be designed to be operated with the loading rate in the range 6 m/h to 9 m/h.

Low filtration rates do not ensure good quality of water; what is more critical is the chemical pretreatment and the filter design. With adequate chemical pretreatment and filter design, filtration rate of up to 15 m/h may be applied without deterioration of the filtrate quality. However, high-rate filtration **(i.e.** 9 m/h < rate < 15 m/h) should be substantiated by a filter column study using the proposed source water.

Filtration rates higher than 15 m/h may be used only in conjunction with the policies and procedures outlined in Section 5.2 - Unproven or Innovative/Alternative Technologies, of the Guiding Principles/Policy Document (1996).

TABLE 6.4

TYPICAL DESIGN DATA FOR DUAL-MEDIA FILTERS

	RANGE	TYPICAL
Anthracite		
depth, mm	300 to 600	450
effective size, mm	0.8 to 2.0	1.2
uniformity coefficient	1.3 to 1.8	1.5
Sand		
depth, mm	150 to 300	300
effective size, mm	0.4 to 0.8	0.5
uniformity coefficient	1.2 to 1.6	1.4
Ratio of coal size to sand size (D_{90} coal/D_{10} sand)	-	3

5. Headloss

The total head available or head loss on a filter is the difference in elevation between the water levels on the inlet and outlet side of the filters. For a specific media and flow rate, the total headloss and the time to reach a fixed headloss depend on the volume of floc retained by the filter. Thus headloss is closely associated with the filter run lengths; operation of a filter with long filter runs and high headlosses will result in break-through of the flocs.

Typical headloss for gravity filtration may be in the range from 0.3 m (clean bed) to 2.5 m (final). The filters may be operated with run lengths between 12 and 72 hours, with 24 hour runs being typical.

Unit Filter Run Volume (UFRV), which is the volume of water produced during a filter run, should be at least 205 m^3/m^2 for successful filter runs. When UFRV exceeds 410 m^3/m^2, there is a declining rate of return on water quality vs. backwash water wastage.

If the headloss at any level in the filter bed exceeds the static head, a vacuum is created, resulting in air binding in the zone of negative pressure. In order to eliminate the problem of negative pressure, the filter should be designed to discharge the effluent water at a level above the sand media surface.

6. Filter Backwash

Three basic methods are available for filter backwash: upflow water wash without auxiliary scour, upflow water wash with surface wash, and upflow water wash with air scour. The suitability of a washing method is related to influent water quality, filtering media and bed configuration, and underdrain design. Consequently, not all washing methods are applicable in all cases, and different methods may or may not yield similar results in a particular case. Declining rate backwash systems are usually not acceptable.

Required backwashing rates are variable and depend on water temperature, filter type and washing method. Water velocity required to achieve the same bed expansion increases as the temperature increases, thus backwash systems should be designed for the warmest wash water temperature.

(i) Upflow Water Wash Without Auxiliary Scour

For the washing to be effective, backwash rate should be sufficient to fluidize the bed with 30 to 40% expansion. The relatively weak cleaning action of water wash without auxiliary scour, generally renders it unsuitable for filters removing large quantities of suspended solids.

When water wash is used alone, a rise rate of 36 m/h to 54 m/h should be applied. Backwash duration may vary between 3 to 15 minutes, depending on how dirty the filter is.

(ii) Upflow Water Wash With Surface Wash

Surface wash is generally applied to supplement upflow backwash where mudball formation is likely to be a problem. Either a fixed-nozzle or rotary wash system may be used; rotary systems are preferred because they provide better cleaning action, lower water requirements, and less obstruction for filter access.

For optimum results, a combined surface and water wash should be designed to be operated in three phases. First, surface wash is activated and operated alone for 1 to 3 minutes. Water wash is then applied simultaneously at a low rate to achieve a bed expansion of less than 10 percent for 5 to 10 minutes. The final phase involves application of wash water only at a higher rate to achieve 30 to 40 percent bed expansion for 1 to 5 minutes.

Rotary surface wash system should be designed to add 1.5 m/h to 6.0 m/h to the wash water flow; and, fixed nozzle system to deliver 6 m/h to 9 m/h.

Because surface wash systems constitute a possible connection between filtered and unfiltered water, backflow prevention devices must be provided in supply lines.

(iii) Upflow Water Wash With Air Scour

There are three approaches to use auxiliary air scour in backwashing filters.

Air scour alone for 3 to 5 minutes followed by low-rate water wash for 5 to 15 minutes could be applied for single media-filters with 0.6 to 1.2 mm media. Air scour should be designed to inject air at 18 to 27 $m^3/m^2.h$; and water wash at 12 m/h to 18 m/h.

The second approach, which is suitable for dual-media filters, is to apply air scour alone for 3 to 5 minutes followed by water wash for 5 to 10 minutes. Bed stratification would occur during backwash cycle. The air scour should be designed to inject air at 54 to 90 $m^3/m^2.h$, and water wash at 12 m/h to 18 m/h.

The third approach is to have simultaneous air scour and water wash followed by water wash alone. During the initial stage, air scour rate of 18 to 90 $m^3/m^2.h$ should be used with a water flow of about 18 m/h for approximately 5 to 10 minutes. This is followed by water wash only for about 5 to 10 minutes at a rate of one to two times that of the previous cycle used with air scour.

6.3.1.6 Flow Measurements

Flow meters should be provided on:

1. the main raw water supply line to measure the total flow entering the plant;

2. individual filters. This may be necessary for constant rate filtration to ensure that each filter receives the same flow; and for declining rate filtration to initiate backwashing of filters;

3. the combined filter effluent line and/or on the distribution main head to measure water usage by the community;

4. the backwash line to measure and check the volumes of water used for backwashing the filters.

6.3.2 Slow Sand Filtration

6.3.2.1 Filter Media

Ideally the effective size (D_{10}) of the sand should be just small enough to ensure a good quality effluent and to prevent penetration of clogging matter to such a depth that it cannot be removed by surface scraping.

The recommended sand size is D_{10} = 0.2 to 0.3 mm with uniformity coefficient (UC) = 1.5 to 2.0. Media having a sand size with $D_{10} \geq 0.3$ mm and UC >2 may be acceptable if a pilot study ascertains that acceptable removals are obtained.

The sand being considered should be washed to remove dust/fine particles to avoid long start-up times. Sieve analysis should be performed and the values plotted to ascertain sand size distribution.

6.3.2.2 Filter Depth

The depth of sand bed is determined by the number of years of operation desired before re-sanding is needed. The years of operation are calculated as follows:

$$Y = \frac{Di - Df}{R.f}$$

in which Y = period of operation, yr
Di = initial sand bed depth, mm
Df = final sand bed depth, mm
R = sand depth removal per scraping, mm
f = frequency of scraping, number/yr

Generally sand bed depth should be between 1 to 1.3 m in depth at the start of operation, a deeper bed may be used if desired. The minimum bed depth should be greater than 0.5 m, assuming the filter is biologically mature. A staff gauge to measure sand bed elevations should be placed on the wall/bank of the filter to denote minimum and maximum sand levels.

6.3.2.3 Filtration Rates

Hydraulic loading rate for the maximum daily design flow may vary between 0.1 m/h and 0.4 m/h. The system should be designed so that when one of the filters is removed from operation for scraping, the combined loading rate of the remaining filters may not exceed 0.4 m/h. While the hydraulic loading rate may vary during the annual cycle and generally increase as the population grows, the flow should be steady over the daily cycle.

6.3.2.4 Headloss

Headloss within a slow sand filter is caused by flow through the schmutzdecke and the sand bed; but mostly through the schmutzdecke.

The clean bed headloss is usually about 0.01 m, depending on the hydraulic loading rate, the temperature and the sand media characteristics. Acceptable headloss limit is economic rather than technical because there is no evidence that higher headloss would cause breakthroughs.

Typical headloss for slow sand filtration may be in the range from 0.1 m (clean bed) to 2.0 m (final). The rate of headloss increase is crucial to the determination of whether slow sand should be selected as the filtration technology because the length of filter runs depend on the rate of headloss increase. This can be analyzed only by pilot plant testing. In general, the rate of headloss increases with time, and the marginal benefits of continuing the filter run decrease rapidly after a certain period.

6.3.2.5 Hydraulics

1. Distribution

Delivery of the entire flow to a filter to one point in the filter bed would result in bed erosion. The consequence of such a condition is short-circuiting of flow through the sand bed. To control bed erosion, the inlet should be designed to distribute and dissipate the kinetic energy. One approach is to distribute the influent flow around the filter bed; the lateral pipe should be large enough that the exit velocity would be sufficiently low. Distribution pipe should be placed approximately 0.3 m from the top of sand bed.

2. Collection

Filters should always operate with a uniform hydraulic loading rate over the sand bed and this is dependent on underdrain collection system.

Collection of the filtered water is through slotted or perforated pipes surrounded by gravel support. The underdrain system should be designed using the manifold hydraulic principle; that is, the headloss within the main pipe should be small compared with the headloss through the orifices in the main pipe. If this principle is maintained, then the hydraulic loading rate across the filter bed would be uniform.

Spacing of the underdrain varies from 1.0 m to 2.0 m; closer spacing is preferred as it would provide an added certainty of uniform loading of the filter.

3. Drainage

To scrape the sand bed, the headwater must be drained to a level just below the sand bed surface. Provision should be made in the design to drain the top portion of the headwater directly either through the influent pipes or through a separate pipe.

4. Backfilling after Scraping

After scraping the de-watered filter must be backfilled with treated water. Provision should be made in the design to backfill the filter through the underdrain system; backfilling from the top can result in entrapment of air bubble which may cause air binding and disruption of the flow.

The treated water may come from an adjacent operating filter or from the clearwater tank. Water from clearwater tank, if used, should be free of chlorine. The headwater should be backfilled to the level of the influent distribution orifices, which is usually placed about 0.3 m above the sand bed to provide the water cushion. Valves and pipings should be provided to accomplish the aforementioned tasks.

5. Flow Control

Flow to the plant should be controlled on the influent side by means of a gate valve located downstream from the influent flow meter. The flow into the plant should be steady over a twenty-four hour period. The treated water storage tank should have an overflow weir with drainage pipeline to waste or recycle, if continuous operation of the filters is desired.

6. Tailwater Control

Tailwater control is necessary so that the water level in the filter can be raised to about 0.3 m above the sand bed immediately after scraping in order to dissipate the kinetic energy of the influent flow.

The design of the system should have a movable weir plate that can be raised or lowered during operation. The initial position of the weir crest, at the start of a filter run, should be about 0.3 m above the top of the sand bed, but the plate should be adjustable enough that it can be lowered to the elevation of the surface of the sand bed once the head loss across the sand bed is greater than 0.3 m.

7. Headloss Measurements

Piezometers should always be installed in filters to measure head-loss. One piezometer should be connected to the headwater above the sand bed, and a second one to the tailwater basin. The piezometers should be 25 to 50 mm in diameter mounted side-by-side, with float balls and scales, to permit easy measurements of water levels.

8. Avoiding Negative Pressures

Negative pressures within the filter bed cause the formation of gas bubbles, which may cause "air binding" and thereby disrupt the flow patterns of water movement through the sand bed.

In order to avoid negative pressures, the system should be designed so that the weir crest for the tailwater elevation control may not be lower than the level of the sand bed during operation.

6.3.2.6 Gravel Support

The function of the gravel support is to support the sand bed and to permit uniform drainage of the overlying sand. In order to have uniform drainage and minimal headloss, the gravel support must be graded with finer material at the top and coarser material at the bottom.

1. Size

The gravel support should be designed so that the top layer of the gravel support should not permit migration of sand from the sand bed, nor should the gravel of any layer find its way to a lower level. The bottom layer should not permit entry of gravel to the underdrain orifices. The following rules should be followed in the design of the gravel support layers:

i. d_{90} (given layer) / d_{10} (given layer) $\preceq$ 1.4
ii. d_{10} (lower layer) / d_{10} (upper layer) $\preceq$ 4
iii. d_{10} (top layer) / d_{15} (sand) $\succ$ 4
iv. d_{10} (top layer) / d_{85} (sand) $\preceq$ 4
v. d_{10} (bottom layer) $\succ$ 2.d (drain orifice diameter)

2. Depth

The thickness of each layer should be greater than three times the diameter of the largest stone. At the same time, as a practical matter, the minimum thickness of gravel layers should be 50 to 70 mm for finer material and 80 to 120 mm for coarser gravel.

6.3.2.7 Flow Measurements

Flow meters should be provided on:

1. the influent side for the whole plant;
2. the influent or effluent side for the individual filters;
3. the volumetric flow meter on the exit side for the whole plant.

The flow meters for the individual filters are used to ensure that each filter receives the same flow and to measure the volumes of water filtered between scrapings. The volumetric flow meter would indicate the amount of water used by the community.

6.3.3 Lead and Copper

Waterworks system should produce non-corrosive water to minimize lead and copper corrosiveness. It is recommended that the owners conduct corrosion control studies unless they can show that corrosion control is already optimized. Corrosion control studies should compare the effectiveness of pH and alkalinity adjustment, calcium adjustment, and addition of a phosphate or silica-based corrosion inhibitor.

The owners should work with AEP to establish a protocol for optimizing corrosion control.

6.3.4 Iron and Manganese Control

Iron and manganese exist in water in both the insoluble and soluble oxidation states. The insoluble iron and manganese are readily removed by filtration. However, the removal of soluble iron and manganese poses a more serious problem, particularly when these metals are organically bound.

Presence of soluble iron and manganese in water may lead to complaints from the consumers as a result of staining of laundry or bathroom fixtures. Water containing iron and manganese also promotes the growth of iron bacteria in mains, with accompanying increases in friction loss.

6.3.4.1 Removal of Iron and Manganese

1. Oxidation/Filtration

Removal of soluble iron and manganese by contact adsorption using pre-coated filter media is the option of choice for treating waters with moderate amounts of iron and manganese in groundwater (<5 mg/L of iron and <1 mg/L of manganese). The filter media that could be used are:

(i) Manganese greensand (natural greensand coated with manganese dioxide);
(ii) Pyrolusite (pure manganese dioxide).

Key to the success of contact oxidation process is the re-generation of manganese dioxide coating on the media, either on a continuous basis or on an intermittent basis.

Dual media filtration is recommended if iron removal is the main objective. Continuous regeneration operation is also recommended where iron removal is the main objective with or without the presence of manganese. This method involves the feeding of a pre-determined amount of an oxidant (potassium permanganate or chlorine), directly to the raw water prior to the filters. If chlorine is used for waters containing ammonia, sufficient chlorine should be fed to go beyond breakpoint to produce free chlorine for regeneration of the media. The oxidant demand should also be determined taking into consideration the presence of DOC and H_2S.

TABLE 6.5

DESIGN CRITERIA FOR IRON AND MANGANESE REMOVAL

Design Parameter	Main Component To Be Removed	
	Iron	Manganese
Regeneration of media	Continuous	Intermittent
Bed type	Dual media	Single media
Depth of bed	Anthracite - 375 to 450 mm MnO_2 media - 450 to 600 mm	MnO_2 media > 750 mm
pH	6.2 to 8.5	7.0 to 8.5
Filter loading rate *	4 to 13 m/h	4 to 13 m/h
Headloss	1.5 m (maximum)	1.5 m (maximum)
Backwash	Sufficient for 40% bed expansion	Sufficient for 40% bed expansion

*a. The higher the concentration of iron and manganese, the lower the loading rates for equivalent run lengths. For optimum design parameters a pilot plant must be operated.

*b. For intermittent regeneration process, higher loading rate of up to 2.0 m/h may be allowed depending on raw water conditions.

Intermittent regeneration is recommended for waters where only manganese or manganese with small amounts of iron is to be removed. This method involves feeding of a pre-determined amount of an oxidant after a specified quantity of water has been treated. Frequently, intermittent regeneration is done during the backwash cycle. Typical design criteria for iron and manganese removal using oxidation/filtration process is shown in Table 6.5.

2. Aeration/Chemically Assisted Filtration

Aeration followed by chemically assisted filtration is effective in treating waters with fairly high content of iron (>5 mg/L) and manganese (>1 mg/L). Without coagulation and flocculation, the oxidized iron may take 12 to 24 hours or more for effective settling, whereas in a properly coagulated water, settling will take place in approximately 2 hours. If manganese is present, pretreatment with one or more oxidants (chlorine, chlorine dioxide, potassium permanganate or ozone) is required.

Design criteria is similar to the requirement outlined in Section 6.3.1 - Rapid Sand Filtration.

6.3.4.2 Control of Iron and Manganese

Iron and manganese concentrations in drinking water may be controlled by blending water from different sources so that the combined iron and manganese concentrations are below the aesthetic objectives stipulated in the GCDWQ.

If iron and/or manganese control with sequestering agents is proposed, the total concentrations may remain greater than the aesthetic objectives. Sequestering is generally ineffective with hard waters; pilot testing may be required to verify the performance of the proposal.

6.3.5 Instrumentation and Controls

6.3.5.1 General

Controls and Instrumentation should be appropriate for the plant size, complexity and number of staff and their skills for each plant. To achieve this, the Designer should develop a control philosophy that will enable the plant staff to effectively monitor and control the plant and major equipment, the treatment process, water production and; plant wastes.

6.3.5.2 Measurement List

For plants of 1 ML/d capacity and greater, the following instruments should be provided as a minimum for the relevant processes listed. For smaller plants, pH measurement and fluoride residual may be by bench testing but all other instruments are appropriate for the relevant processes listed.

1. **Raw Water Instrumentation**
   - Low-level switches to shut down the raw water pumps. These should be hard-wired to the starters.
   - Running and trip indication for raw water pumps
   - Raw water turbidity, pH, pressure, flow rate, and flow volume

2. **Rapid Mixer**
   - Running and trip indication

3. **Flocculators**
   - Running and trip indication
   - Speed if variable speed type

4. **Solids Contact Clarifiers**
   - Recirculator speed indication
   - Running and trip indication
   - Level indication
   - Blow down valve status
   - Turbidity and pH following clarification

5. **Softening**
   - If lime softening is used, pH following recarbonation
   - Recarbonation CO_2 feed status

6. **Filter Instrumentation**
   - Turbidity on each individual filter effluent and filter to waste. This can be a single instrument for each filter if piping arrangement permits.
   - For constant rate filters: differential head loss across the filter media
   - Filter flow rate

- Where the backwash sequence is automated, provide open and close limit switches or position on all filter valves and status on backwash equipment.
- Filter run time

7. Backwash Instrumentation

- Running and trip indication for backwash pump(s)
- Running and trip indication for air blowers (if air scour is used)
- Backwash flow rate and flow total

8. Clearwell & Distribution Pump Instrumentation

- Level indication for clearwell and other tanks
- Low-level switches to shut down the distribution pumps. These should be hard wired to the motor starters.
- Turbidity, chlorine residual, fluoride residual (if fluoridation is practised), pH, pressure, flow rate, and flow total on plant discharge.
- For variable speed pumps, indicate the pump speed

9. Chemical Systems

- Running and trip indication for chemical loading, batching and pumping equipment.
- Low and high level indication in storage bins, silos or tanks
- Level indication for tanks
- Weigh scales for hydrofluosilicic acid day tanks or storage if no day tank is used
- Weigh scales for gaseous feed chemicals such as chlorine or sulphur dioxide
- Speed indication on variable speed pumps
- Rotameters for carrier water feed systems
- Chemical feed flow rate is desirable but not mandatory

10. Miscellaneous Instrumentation

- Run time meters on all pumps and major electrically driven equipment.
- Speed, run time, oil pressure and temperature gauges, fault signal switches and manual start and shut down on engines
- Where the plant is automated or operated remotely from either within the plant or outside, provide open and close limit switches or position on all major valves, status on all major equipment and security instruments including door switches, building temperature switches and smoke alarms.
- Any additional instrumentation recommended by equipment manufacturers.

6.3.5.3 Degree of Automation in Plant Control

The control system may be manual or automatic or a combination thereof. Regardless, the system should be designed to promote energy efficiency, conserve water, and reduce waste while meeting the treated water quality standards and demands under all anticipated conditions. Discuss the operating philosophy with plant staff and owners to determine the appropriate degree of automation.

In the case of a manual system, all equipment is started and stopped by the operator, all backwash sequences and other process operations are controlled by the operator, and chemical and pump rates are manually adjusted. This requires that the plant be manned continuously while in operation, perhaps with more than one operator.

In the case of an automatic system, all equipment is started and stopped by the control system, with chemical feed rates and pump rates adjusted automatically to maintain the system levels, discharge pressures, etc. This may allow un-attended plant operation or operation with a single operator, but requires a more complex and expensive control system, with associated maintenance. Provide the ability to manually operate all equipment.

For systems with rapidly varying raw water conditions, fully automated plants should not normally be considered.

6.3.5.4 Alarms and Status Indication

All alarms must be latched until the Operator has acknowledged them. If the alarm is indicated by a lamp, it must flash until acknowledged then remain steady until the alarm clears. If it is indicated on a computer screen, an appropriate color code or symbol must be used to indicate for each alarm whether it has been acknowledged. Automated systems should log the time at which the alarm occurred, the time it was acknowledged and the time it cleared. Logs may be printed on paper or recorded electronically.

Valve and equipment status should use a consistent method of symbols and colours, whether the status is indicated through lamps or on a color computer screen. The color-coding scheme should be consistent with any existing equipment displays elsewhere in the plant.

As a minimum, the following alarms should be provided:

- High turbidity on the raw water, clarifier effluent (if applicable), filter effluent, and plant discharge
- High and low pressure on the raw water line
- High flow rate on the raw water line
- High and low level in clarifiers or flocculators
- High torque on solids contact clarifier recirculator and rake
- High torque on flocculators
- High level in filters
- High and low level in chemical storage tanks
- High and low chemical feed rates
- High flow rate on each filter individually (also low flow rate on declining rate filters)
- High and low levels in each clearwell, pumpwell, and reservoir
- High and low pH on the raw and treated water (if on-line measurements are provided)
- High and low chlorine residual on the plant discharge (where on-line measurements are provided)
- High head loss on the filters (if constant rate type)
- Trip or failure to run on each pump
- High and low pressure on the plant discharge line
- High flow rate on the plant discharge line
- Chlorine gas detection in the chlorine storage, metering and injector rooms.
- Chlorine scale low weight (where scales are equipped with transmitters)
- Valve operation failure (where valves are provided with limit switches)

More alarms may be required where additional treatment processes are provided. Alarms should be provided for all control system interlocks that can shut down equipment or systems. In plants that are left unattended for periods of time, an automatic alarm dialler should be provided.

6.3.5.5 Control Equipment (Automatic Systems)

Automatic systems should use either Programmable Logic Controllers (PLC's) or a Distributed Control System (DCS). The operator interface may be in the form of traditional control panels (ie: lights, gauges and switches), electronic control panels (with text and/or graphics) and computers.

Digital communication between components of the control system must be reliable and self-monitoring. The communication protocol must meet the following requirements:

- It must include error-checking and reporting, to ensure that data is correctly transferred from one component to another.
- The components of the system must detect the failure of the communication system (either between individual components of the system or between the system and the operator).
- It must be compatible with a variety of manufacturer's instruments and equipment, in order to allow for expansion of the system.

If a DCS is provided, the communication protocol will be proprietary and the manufacturer should be consulted regarding reliability, error-checking, and the possibility of connecting other manufacturer's equipment to the network.

If PLC's are used, the communications protocol should use one of the widely accepted industry protocols such as, but not limited to: Modicon MODBUS, Allen-Bradley DATA HIGHWAY and TCP/IP Ethernet.

The operator interface may consist of a local hard wired control panel or mimic, character based input/output panel, personal computer or workstation depending on system size, process complexity, control system functions and operator interface manufacturer. Where personal computers or workstations are used, select the hardware based on reliability, software compatibility, vendor support and suitability for continuous operation in the plant environment. The operator interface software may provide the operator with interactive control and monitoring of the plant, handle and annunciate alarms, log and trend events and process variables and generate the required reports. Process control and logic should be performed by the PLC or DCS and not the operator interface computer or workstation.

6.3.5.6 Field Instruments

1. Level Instruments

Where access to the top of the reservoir is convenient (such as in a clearwell), ultrasonic level transmitter should be used. Where access to the bottom of the reservoir is convenient (such as at a tower or above-ground reservoir), a pressure transmitter should be used as a level-sensing device.

(i) Ultrasonic Level Measurement

The ultrasonic level transmitter fires a “sonar” signal toward a surface, such as the surface of the water in a well, and measures the time required to receive an “echo” in order to determine the level of the liquid.

The ultrasonic transducer should be installed so that it is protected from damage, there are no obstructions between the transducer and the water surface, and it is accessible for calibration and maintenance.

- The transducer should be installed in the top of a stilling well to prevent turbulence from producing errors in the reading. The stilling well should be a continuous length of pipe, either PVC or steel of sufficient diameter and without couplings or fittings that could reflect a sonic echo back to the transducer thus giving a false reading.

- The well must extend from a convenient height above the high-water line, at which level the transducer will be installed, to the low-water line.

- Consider the transducer’s “blanking distance”, inherent its design, and ensure that the transducer is mounted high enough above the high-water line so that it will properly read the highest water level anticipated.

- Several holes should be provided in the side of the stilling well near the bottom for water to enter the well. The holes must be large enough to prevent clogging if silt is present.

- The controller and display should be located where it can be conveniently read by the operator.

- Where the air temperature between the transducer and the liquid surface is not constant (this is usually the case), provide a temperature measurement for the controller in order that it can compensate for the speed of sound travel through the air, and correct for temperature variations. Note that some manufacturers include the temperature sensor in the transducer itself, while some provide a separate temperature probe.

(ii) Pressure-Sensing Level Transmitter

The pressure-sensing level transmitter reads the head of a column of liquid and transmits a signal proportional to the level of liquid.

- The level transmitter should be installed as near as practical to the bottom of the tank being measured, so as not to introduce a zero offset in the reading.

- A block and bleed valve should be provided on the pressure line so that the transmitter can be calibrated for zero level, and can be removed from service.

- If the pressure-sensing line is small in diameter (12 mm or less), clamp it to supports or walls to provide adequate support.

- If the transmitter is equipped with an integral display, the transmitter should be located so the display is clearly visible. If no display is provided, and the head being measured is high enough (100 kPa or higher), consider installing a pressure gauge in addition to the transmitter as a backup and calibration aid.

2. Flow Instruments

On line, flow meters should generally be one of the following types:

- Turbine (or nutating disk)
- Magnetic
- Ultrasonic (either transit-time or Doppler)

All of these types of instruments can be equipped to provide both flow rate and flow total measurements.

Price, line size, flow rate, flow range, required accuracy and water quality will dictate the selection of the type of instrument. The following are some general guidelines:

- Where considerable silt is present (as in many raw waters), either a magnetic or a Doppler-type ultrasonic meter should be used. Turbine meters will wear rapidly and are not practical. Transit-time meters may not operate properly if there is considerable silt (consult the manufacturer).

- An ultrasonic meter is generally more economical than a magnetic type on lines of 300 mm diameter and larger.

- For, on lines of very low flow rate (less than 0.3 m/sec), turbine or magnetic flow meter is recommended. Where chemicals are present in the water, check with the manufacturer to ensure that the meter will not suffer damage. Corrosion- and abrasion- resistant linings should be considered for these applications. Regardless of the meter type, the minimum flow velocity should be within the specified range of the meter.

- For, on lines of high flow velocity (higher than 5 m/sec), magnetic or ultrasonic flow meter is recommended. Regardless of the meter type, the maximum flow velocity should be within the specified range of the meter.

- Where the water is free of solids and bubbles (as is the case on a potable distribution line), Doppler-type ultrasonics will not operate; a transit-time type should be used.

(i) Turbine Flow Meters

Turbine flow meters determine the flow rate by reading the rotating speed of the turbine, which is immersed in the fluid. A flow totalizer is almost always included, and a flow transmitter is usually available. Because they totalize volume without power, they will continue to operate during power failures and because they will operate without any configuration on the part of the user or operator they are often used where ease of use and maintenance are essential.

- Where debris may be present in the water, such as in a raw water intake, a screen filter (such as a Y-type strainer) upstream of the meter should be provided.

- A continuous straight run of piping upstream of the meter, 10 pipe diameters if possible should be provided, to produce a smooth flow profile through the meter. This will minimize errors in the reading.

- If it is not possible to provide at least five diameters of straight piping upstream of the meter, install straightening vanes in the pipe immediately upstream of the filter. Even with vanes, the accuracy of the meter may be compromised.

- Five diameters of straight piping should be provided downstream, if possible.

- If a totalizer or flow rate display is provided, they should be located so that the display is easily read.

(ii) Magnetic Flow Meters

Magnetic flow meters operate by applying a magnetic field around the flowing liquid and reading the voltage produced on a pair of immersed electrodes.

- The manufacturer should be consulted regarding electrode material and liner material. The meter should operate with silt, chemicals, etc.

- It is essential that the pipe be full of water at all times; the meter will not operate with large air bubbles in the pipe. Some small bubbles, such as are found downstream of pumps, can be tolerated.

- A continuous straight run of piping should be provided upstream of the meter, 10 pipe diameters if possible, to produce a smooth flow profile through the meter. This will minimize errors in the reading. Because magnetic meters read the total voltage produced across the full width of the pipe, some averaging is provided, and this makes them more resistant to turbulence than either turbine or ultrasonic meters. Less than 10 pipe diameters straight run upstream may compromise the accuracy of the meter.

- Five diameters of straight piping should be provided downstream if possible.

- Magnetic meters should be supplied with flow rate and flow total displays; the controller should be installed so that the display is easily read.

(iii) Ultrasonic Flow Meters

Ultrasonic flow meters operate by firing a sonic “pulse” through the pipe wall into the flowing liquid. A transit-time meter uses two transducers, one mounted upstream of the second, and measures the difference in travel time for a pulse from one transducer to the other. A Doppler type measures the difference in the frequency received by the transducer as the sonic pulse reflects off particles or bubbles in the liquid. In either case, the difference is directly proportional to the velocity of the liquid.

- The manufacturer should confirm that the flow meter will operate with the pipe wall material and thickness expected.

- Ultrasonic flow meters should not be installed where the pipe will contain large bubbles or air pockets; the sonic pulse will be disrupted so that the meter won’t operate.

- Ultrasonic meters are sensitive to the flow profile; at least five pipe diameters of straight piping should be provided (ten pipe diameters recommended) between the meter and an upstream elbow or other hydraulic disturbance.

3. Water Quality Instruments

The most frequently-used water quality measurements are turbidity, pH, and chlorine residual. On-line turbidity measurement is relatively inexpensive and should be provided in any plant, on the raw water, flocculator or clarifier effluent (if applicable), each filter effluent, and final plant discharge lines. In larger plants, on-line pH and chlorine residual are generally used, but these can be done though lab tests in smaller plants.

(i) Turbidity Instruments

Turbidity instruments usually measure the degree to which a beam of light is transmitted or scattered as it passes through a sample of the liquid being measured. A small constant flow of liquid is required to pass through the turbidimeter. It is important that the liquid be free of bubbles which would scatter the light and produce an erroneously high reading.

- A transmissive type flow meter should be used for low-turbidity applications such as treated water (turbidity range 0-100 NTU) and a surface-scatter model for high-turbidity applications such as raw water (range 0-5000 NTU).

- A needle valve and rotameter should be provided to adjust the flow rate through the turbidimeter so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flow rate adjustment easier.

- The liquid stream is not affected by the turbidimeter; it may be returned to the process or discharged to waste.

- The sense element should be located as near to the sample point as practicable to minimize lag time. Where the water contains settleable material, the sample line velocity should be high enough to prevent sedimentation in the line. The use of clear piping should be avoided to reduce the possibility of algae growth.

- Where a sample line may become plugged by silt, as in a raw water measurement, provide a manual flush valve should be provided with pressurized plant water to flush the silt either backward into the process line or to waste, as required. A block valve should be provided for the turbidimeter to protect it from the high-pressure flush water.

- The sense element may be mounted some distance from the controller. The controller should include a display and should be installed so that the display is easily read.

(ii) pH Instruments

pH is read by the measurement of an electric potential generated at a pair of electrodes which are wetted by the sample stream. All pH instruments use a buffer solution, which is generally pumped to the electrodes in very small volumes by the controller. The solution must be replenished at intervals.

- A needle valve and rotameter should be provided to adjust the flow rate past the electrodes so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flow rate adjustment easier.

- If the sense element may become clogged with silt, a filter should be provided upstream. The sense elements are generally very fragile, so flush lines should only be provided where the electrodes can be completely removed from service during flushing.

- The liquid stream is contaminated with buffer solution during the measurement; the stream should be discharged to waste.

- If the controller includes an alarm contact to warn of low buffer solution level, the contact should be tied into the alarm system to remind the operator to refill the controller.

- The sense element must not be mounted far from the controller because of the very low-level signals involved. If necessary, the sample line should be routed to a location where the sense probe and the controller may be located near each other.

- The controller should include a display, and should be installed so that the display is easily read.

(iii) Chlorine Residual Instruments

Chlorine residual measurements fall into two categories: amperometric, which measures a potential generated at three electrodes, and polarographic, which measures a colour change when an indicator is added to the liquid sample. Both types of instruments require periodic refilling with buffer or indicator solution.

- A needle valve and rotameter should be provided to adjust the flow rate past the electrodes so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flow rate adjustment easier.
- The liquid stream is contaminated with buffer solution during the measurement; the stream should be discharged to waste.
- If the controller includes an alarm contact to warn of low buffer solution level, the contact should be tied into the alarm system to remind the operator to refill the controller.
- The sense element must not be mounted far from the controller because of the very low-level signals involved. If necessary, the sample line should be routed to a location where the sense probe and the controller may be located near each other.
- The controller should include a display, and should be installed so that the display is easily read.
- Because chlorine measurements are usually limited to treated water lines, it is not necessary to install flush lines or filters to protect the instrument from debris.

4. Pressure Instruments

Pressure may be simply indicated on a gauge or transmitted (and optionally indicated as well) by a transmitter.

(i) Pressure Gauges

Pressure gauges are available to read both differential and single-ended pressure. By far the most common measurements are single-ended, although differential gauges are used to read head loss on water and air filters.

- Where a pressure gauge is reading a pressure produced by a pump (normally required) the gauge should be protected from vibration by filling it with either silicone liquid or glycerine. Silicone should be used if the ambient temperature will fall below -30°C.

- The range of the gauge should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the gauge should not be operated full-time near the top end of the scale. This will provide some safety margin on over-pressure as well as prolonging the life of the gauge.

- The gauge should be installed where the lens will not get damaged and where it can be read easily. Choose a gauge with a top-mounted stem where it will be installed near the ceiling so that the dial will read right-side up.

- For water applications (both raw water and treated) a bronze or 316 stainless steel bourdon tube mechanism should be used. For applications on chemical lines, the manufacturer should be consulted for compatibility between the process and the gauge material.

- On corrosive liquids and processes containing solids, or where the gauge material is not compatible with the process, an isolating diaphragm should be used between the process sense line and the gauge to protect the gauge.

- A block and bleed valve should be installed between the process and the gauge, or between the process and the diaphragm, to take the gauge out of service.

(ii) Pressure Transmitters

Pressure transmitters are available to read either differential or single-ended pressures. The single-ended type may read either gauge pressure (the pressure relative to the atmosphere) or absolute pressure (relative to a vacuum). Absolute pressure measurements are not common. Differential measurements are commonly used to determine when a filter needs washing.

- The range of the transmitter should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the transmitter should not be operated full-time near the top end of the scale. This will provide some safety margin on over-pressure as well as prolonging the life of the sense element in the transmitter.

- For water applications (both raw water and treated) a bronze or 316 stainless steel sensing diaphragm should be used. For applications on chemical lines, the manufacturer should be consulted for compatibility between the process and the diaphragm material.

- On corrosive liquids and processes containing solids, or where the sense diaphragm material is not compatible with the process, an isolating diaphragm should be used between the process sense line and the transmitter to protect the transmitter.

- A block and bleed valve should be used between the process and the transmitter, or between the process and the isolating diaphragm, to take the transmitter out of service, and to facilitate calibration.

- Where a pressure gauge is not installed on the same line as the transmitter, an integral display should be provided on the transmitter for local indication.

6.3.5.7 Process Controls

1. Pumping Systems

(i) General

Regardless of the function of the pumping system, its control will normally be achieved through monitoring level, flow and/or pressure. The choice of control parameter(s) will depend on the system's function and features. Controls and monitoring for the following systems is discussed:

- Raw Water Pumping
- Finished Water Pumping

(ii) Raw Water Pumping

Raw water pumping is normally controlled by flow, since this sets the production rate of the treatment processes. Typically, this is achieved manually, by selecting the number of raw water pumps operating. If there are variable speed pumps, these will be controlled by flow; with their speed, and output, controlled to match a manually selected flow set point.

Alternatively, pumps may be controlled by level in the plant's treated water clearwell storage reservoirs; or in one of the open unit processes. At selected level set points, falling water level will bring another pump, rising water level will shut down a pump. If variable speed pumps are used, an analog level signal can be used to control pump speed, and hence its output -- to maintain water level within a selected operating band.

Raw water pumping rate should be varied gradually, if possible, and only when necessary. Flowrates through the treatment processes should preferably kept steady, as this will achieve better and more consistent treatment and water quality. This may be achieved by setting the raw water pumping rate, and hence the plant production rate, to meet the anticipated demand for the day.

On the suction side of the pump(s), pressure is monitored by level or pressure indicator. The pressure/level measuring device will initiate low level (or pressure) alarm, and low-low level (or pressure) alarm and shut down of pumps, to protect the pumps from cavitation damage or running dry. Where the raw water source exhibits or is subject to rapid increase in free surface elevation (F.S.E.), the pressure/level measuring device should initiate high level alarm and raw water pumps and possibly the whole plant should shut down for flood protection.

Flow monitoring may be provided on either the suction or discharge side of pumps; normally on the discharge side. Flow monitoring should indicate flowrate, and accumulated flow volume. Coagulant and pre-disinfectant (if used), should be flow paced to the flowrate signal.

Pressure on the pump discharge should be monitored. The combination of flow and pressure will serve to monitor pump performance. If the pump discharge flowrate is to be controlled by a modulating valve, pressure should be monitored upstream of the valve to ensure pumps are operating within the normal process operating range, and also to ensure they are not operated outside their allowable envelope. High pressure and low pressure set points should be provided to initiate an alarm condition; high-high and low-low pressure set points will initiate pump shut down. Likewise, monitoring the valve position helps to ensure that the valve can be operated within its working range.

Flow splitting is required, where two or more process trains are used. This may be through separate flow meters and flow control valves or through flow splitter boxes employing weirs. In the latter case, an adjustable weir on valve should be provided in order that the flow splitting can be balanced or adjusted.

(iii) Finished Water Pumping

Finished water pumping control should ensure that varying demand from the distribution system can be met while maintaining adequate pressure in the distribution system. This will be achieved by controlling flow or pressure, depending on the distribution system into which it feeds.

Flow control may be used in larger systems when the control system is essentially manual, and the distribution system has sufficient storage to accommodate the difference between varying demand and the selected pumping rate. This essentially comprises manual selection of the number of operating pumps, and will require continual operator monitoring and supervision or an automatic system to ensure distribution storage is not depleted or overflowed.

Monitoring discharge pressure is a common approach in controlling small and medium systems. Pump discharge pressure set points will start or stop pumps in a pre-selected sequence. Increased demand in the distribution system will result in falling pressure, when pressure reaches the low pressure set point, it will initiate starting the next duty pump, thereby restoring pressure to within an acceptable range. If pressure continues falling and again reaches the low pressure set point, the next duty pump will start, and so on.

On rising pressure, a pump will be shut down when the high pressure set point is reached. If pressure continues to rise again, the next pump in the sequence will drop out. The pressure set points must be selected to ensure distribution system pressures remain within acceptable limits; the pumps must be selected to ensure they can operate over the range of the operating set points.

Variable speed pumps may be used in finished water pump systems. Multiple pumps are still needed, but fewer units can be used to cover the same flow range, if some or all are variable speed. The most important advantage of variable speed pumping is the ability to maintain a constant discharge pressure into the distribution system. Control of variable speed pumps will be by pressure. Pump speed and output will vary in response to a drift in discharge pressure from the selected set point. A drop in pressure below set point will initiate incremental pump speed increases until pressure set point is restored. A rise in pressure above set point will initiate incremental pump speed decreases until pressure set point is restored. When maximum (minimum) pump speed limit is reached and pressure set point has still not been restored, another pump will start (stop), and the variable speed unit will ramp down (up) until pressure set point is restored. Correct selection and sizing of the pumps is vital to ensure the speed of the pumps remains in the recommended range.

Regardless of the control system, pressure (or level) should be monitored in the suction side to provide alarm and shut down or low pressure (level) and low-low pressure (level).

Discharge flowrate should be monitored continuously, and the accumulated volume recorded.. Flowrate will be used to control the feed rate for secondary disinfectant, and where applicable, corrosion control chemicals, and pH control chemicals. Discharge pressure monitoring will also provide alarm on low or high pressure, and pump shut down on low-low or high-high pressure.

2. Treatment Processes

(i) Travelling Screens

Two methods may be used to control the operation of travelling screens:

a. Simple manual start/stop which requires the presence of the operator at the screen in order to start and stop the screen. This method is not recommended where sudden changes in raw water quality could result in heavy debris accumulation on the screens.

b. Automatic activation by differential level or time. This method uses the differential level across the screen to provide the start condition. The screen should run at least one complete screen cycle before stopping. The screen may be programmed to stop when the differential level is returned to the clean screen value, the final stop should be controlled using a sensor to determine cycle completion (ie. limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the travelling screen.

(ii) Chemical Feed Systems

a. Liquid Chemical Feed

The chemical dose rate should be flow paced to the plant flow in the part of the process that the chemical is to be injected into. Two methods are typically used to achieve this: metering pump, or flowmeter and flow control valve on the chemical feed.

A. Metering Pump Feed Control

Positive displacement type (diaphragm, peristaltic or progressive cavity) pump should be used. The output of the pump is directly controlled by a 4-20 mA signal from the flow transmitter on the plant flowmeter. On plant shut-down, the flowmeter (usually the raw water flowmeter) will signal the metering pump to stop and a solenoid on the dilution water to close. A load cell or pressure (level) transmitter on the chemical storage tank should provide warning signals when chemical supply is low, and should have alarm and initiate plant shut-down on low-low level.

B. Flow Meter Control

Where the need, and justification for a more accurate and positive control system exists, flow meter control may be provided. Where the chemical feed rate is controlled by an in-line flow control valve. A PLC receives a 4-20 mA signal from the flow transmitter on the plant flowmeter. Using this dose rate set point, the controller will look at the flow rate on the chemical feed flow transmitter and signals the in-line flow control valve to a position that will control the feed rate to the established set point.

On plant shut-down, the controller will signal the in-line control valve to close. Depending on the range of feed rates, multiple flow meters and control valves may be required.

As with the metering pump system, low and low-low level alarms/shut-down should be provided on the chemical storage tank(s).

b. Dry Chemical Feed

Dry chemical feed systems typically include a packaged bulk storage combination feeder and mixer. The feeder can be gravimetric or volumetric, and will be controlled by a 4-20 mA signal from the flow transmitter on the plant flowmeter.

The chemical feeder discharges to a dissolving tank where it is mixed with plant service water to form a solution (slurry) suitable for dosing. Plant service water flowrate is manually set and monitored by flow indicator. The rate needs to be suitable for the range of anticipated chemical feeds and its solubility; the rate may need to be manually adjusted seasonally. For a specific plant service water flowrate, the variation in chemical feed rate creates a corresponding variation in solution strength fed to the process water.

The solution is fed via an hydraulic injector (if into a pipeline under pressure), or directly by gravity into open channels or tanks. On plant shut-down a signal from the raw water flowmeter will call for the dry feeder to stop and a solenoid valve on the plant service water feed to the dissolving tank to close. If an injector is used, a solenoid valve on the plant service water will also close.

(iii) Rapid Mixing

Rapid mixing of coagulant and other chemicals, is achieved by mechanical or hydraulic mixing. Mechanical mixing may be in-a tank or in a pipe and will comprise of a propeller or impeller assembly, usually electric motor driven. Control of the rapid mixer will be simply on or off; the unit should operate continuously whenever the plant is producing.

The use of variable speed or two speed drives can be used to vary mixing energy. If used, the speed may be controlled manually, based on operating experience and/or varied proportion to the flow rate in order to reduce backmixing if this is of concern.

Hydraulic rapid mix can be in-pipe (static mixer or pumped jet) or free-fall (weir or cascade). These systems require no controls other than on/off for the pumped jet. With the static mixer, it is useful to indicate head loss across the mixer to confirm energy level and also monitor for headloss build-up which could be indicative of chemical accumulation.

(iv) Flocculation

Flocculation is achieved mechanically or hydraulically in tanks. Mechanical flocculation requires paddles, picket fences or turbines to gently mix the coagulated water to produce a settleable floc for subsequent settlement, (or if the liquid - solid separation process is flotation, to produce a fine pin-point floc for floating). Some processes, such as solids contact clarifiers, integrate flocculation into the unit process. Flocculation requirements should be addressed in terms of the unit process parameters.

Where flocculation is a discrete unit process, it is normally achieved in multiple tanks operating in trains of usually two or three stages. The mixing energy level should be optimized for the water being treated; and will vary as the water quality changes. Jar testing or streaming current monitor will indicate the appropriate chemical dose. Jar testing will also indicate the mixing energy required for optimum flocculation.

For mechanical flocculation, variable speed drives should be used on the equipment. This will allow mixing energy to be adjusted to optimize the process. Speed control will be manual, based on jar tests and operational experience.

It is important that floc size and distribution through the floc cells be observed daily, along with a check of mixer speed. No other control or instrumentation is necessary, other than the status normally required for the motor drives.

With hydraulic flocculation, mixing energy is a function of head loss (and thus flowrate) through the floc cells. Thus, the only means of controlling hydraulic flocculation is to vary the flowrate. Control of the process is limited to manual control of flowrate, or where applicable, manual manipulation of valves to control the flow through the process to increase/decrease retention time.

(v) Clarification

Careful monitoring and control is most important to successful clarification. Adequate instrumentation to measure water quality parameters prior to and after clarification is essential.

a. Sedimentation

Sedimentation can be accomplished in horizontal flow, upflow or upflow solids contact clarifiers. The latter combines sedimentation with the chemical coagulation and flocculation processes.

For all the types mentioned, provisions should be made to observe the clarification process. These observations should focus primarily on the floc condition throughout the tanks; poorly settling floc, a change in floc size, flow carry over onto the filters are all indicators that the clarification process may not be optimal. Jar testing should be undertaken to determine if the coagulant (and polymer) dosage, flocculation energy or flow rate needs adjusting.

The operator should watch for floating sludge; algae growth on tank walls and launders; and any abnormal appearance in the process water.

For horizontal or upflow clarifiers that are equipped with mechanical sludge removal, it is necessary to monitor sludge quality and consistency. For smaller plants this may be done manually by drawing daily samples and noting the concentration, texture and condition of the sludge. Provision should be made for this sampling. In larger installations, ultrasonic or magnetic sludge density meters should be provided. Sample points should be located at strategic depths in the sludge hopper to provide information on sludge accumulation rate. An alarm should be provided to indicate failure of the mechanical sludge removal equipment.

Information gathered from this type of monitoring plus the operator's experience will be used to maintain control of the process. Provision should be made for the operator to adjust the sludge sweep cycle, speed and duration.

Upflow solids contact clarifiers are more complex to operate because they rely on maintaining a sludge blanket to develop and capture the floc from the rising flow. Although a very efficient clarification process, it is sensitive to hydraulic and solids shock loading and should preferably be run continuously at as steady a flowrate as possible.

Starting or restarting a solids contact clarifier (SCC) requires establishing the sludge blanket. This may take some days on initial start-up, less on restarting an operational unit, depending on the sludge condition and quality.

Monitoring of the sludge blanket position, depth and condition once it is established is critical to the operation. Typically, this can be done through manual sampling form tappings at various strategic depths within the tank. Again, the use of sludge density monitors should be used in larger installations to provide continuous monitoring of and data on the sludge blankets.

With the proper level of monitoring and the operator's experience, the SCC process variables can be controlled. These including recirculation turbine speed, sludge scraper speed (where applicable), sludge blow down cycle (frequency, duration), and chemical feed rates. Turbine speed and sludge scraper speed are normally adjusted manually to suit operating conditions. The sludge blow down cycle can be initiated by the volume of water processed, a timer and/or sludge level. Again, the volume, timer and/or sludge level set points will be manually set by the operator based on raw water conditions, sludge accumulation rate and sludge condition.

b. Dissolved Air Flotation

The process variables in DAF are:

- flowrate
- recycle rate
- float removal cycle

The DAF process comprise multiple tanks operating in parallel. To handle varying flowrates, the operator should to the extent possible match the number of units operating to the demand. The process can be shut down and restarted very easily, with the effluent quality being restored quickly (within 10-20 minutes). Shutting down units during low demand keeps the operating units performing optimally, and also reduces the amount (and cost) of recycle.

Recycle rate is set based on raw water quality and operating experience. Recycle flow is controlled by the recycle pumping rate and the nozzles through which the recycle is released into the DAF tank(s). Pressure in the recycle system upstream of the nozzles should be kept in the range 450 to 725 kPa to maintain the air in solution and provide the proper micro sized bubbles on release into the tank. Recycle flowrate can therefore be varied, provided it does not cause the pressure to fall outside the design limits. Recycle rates may be further varied by shutting down bank(s) of nozzles and/or adjusting nozzles (if applicable).

The saturation process can use either injectors or a packed bed saturator. Injectors discharge saturated water into a pressure vessel; with the packed bed saturator, the saturator itself is a pressure vessel. Within this pressure vessel, the level is monitored; this level controls the recycle system.

Variable speed recycle pumps, controlled by level in the saturator tank, are used for packed bed systems. When banks of nozzles are opened/closed, the resultant recycle flowrate change initiates a pump speed adjustment to restore the level set point. Depending on the number of pumps and control system, additional recycle pumps are brought on-line or dropped off in response to saturator tank level.

Alternatively, with injector or small packed bed systems, dedicated fixed speed pumps may be used. Each injector or tank would have its own recycle pump which would operate whenever its DAF train comes on-line.

Float removal may be continuous or intermittent, and can be accomplished mechanically and/or hydraulically. Alarms should be provided for equipment failure or if water levels fall outside of limits.

There are various systems, some proprietary, used for water clarification. The control systems for each will not be described here. The following paragraphs provide general guidance on the recommended monitoring and control for float removal, and should be read in conjunction with manufacturer's operating instructions, where applicable. Provisions must be made for the float to be observed; this is necessary to ascertain if chemical feed and/or flocculation energy need adjustment.

Continuous float removal is achieved by a mechanical sludge skimmer or paddle, pushing float over a weir (beach). Water level has to be maintained relatively constant for the float removal system to operate efficiently. Level is controlled either by a tank effluent weir, which is set for the design flow, and should not require adjustment unless conditions change (e.g. settlement of tankage, new design flow etc.), or by an effluent control valve with a level controller. The float removal system variables (travel/rotation speed) should have the capacity for manual adjustment.

Intermittent float removal typically utilizes hydraulic methods. Water level in the DAF tank is raised, by restricting the effluent flowrate, to allow float to discharge over a weir. When the float has been discharged, the effluent flowrate is restored and water surface drops to its normal operating level. The float removal cycle (frequency and duration) should have the capacity for manual adjustment.

(vi) Filtration

Filtration is the most critical stage in the particulate removal process. It needs to be monitored and controlled closely to ensure treated water quality is consistent and within guidelines.

Two types of filtration are used for water treatment:

- rapid gravity filtration; and
- slow sand filtration

a. Rapid Gravity Filtration

The majority of municipal water filtration plants use rapid gravity filters. Rapid gravity filters (RGF) are operated in one of two ways: constant rate and declining rate. The control systems for each are different. Each is described below.

A. Constant Rate

Flow through a constant rate RGF is controlled by a flow control valve on the filter effluent or by influent flow splitting and filter level control. For the flow control type, the effluent valve position is controlled by a flowrate signal from a flow meter, usually located on the filter effluent. For the level control type, the effluent valve position is controlled by the water level in the filter.

A filter run will be terminated, and the bed backwashed on one or any of the following:

- run time
- headloss across the bed
- effluent turbidity
- effluent particle count (optional)

The termination of a filter run and start of a backwash cycle can be initiated automatically or manually. Plants that are manned continuously have this option. Plants that are not manned all the time should be designed for automatic initiation, with provision for manual override; this will ensure filters are backwashed when required, even if the operator is not there.

At minimum, both headloss and effluent turbidity should be monitored on each individual filter. Headloss is monitored by measuring the differential pressure between the effluent line (upstream of the control valve) and top of the filter media. The pressure signal will initiate an alarm on high level, and in a non-continuously manned plant, initiate a filter backwash cycle. On high-high level, the filter should shut down until the operator has investigated the cause of the high-high alarm and/or manually initiated a backwash.

Turbidity should be monitored by an on-line turbidimeter -- one for each filter. As with headloss, a high turbidity set point can initiate an alarm (and possible backwash cycle); and a high-high turbidity set point should shut down the filter to prevent poor quality water reaching the clearwell.

The turbidimeter should be located upstream of the filter-to-waste diversion point or a second turbidimeter provided for filter-to-waste if the piping is separate. Filter-to-waste duration should be based on the turbidity.

Continuous recording of effluent turbidity is required in assuring filter and plant performance.

B. Declining Rate

Flow through a declining rate RGF is not directly controlled as is the case with constant rate RGF. The rate simply decreases as the filter plugs. An effluent valve with manually adjustable stops is set to ensure the flowrate through a clean bed is not excessive. Once set, this valve will return to the set position after backwash (or after being closed for maintenance etc).

A filter run will be terminated on one or any of the following:

- run time
- effluent flowrate
- effluent turbidity
- effluent particle count (optional)

With declining rate control all filters have the same driving head, hence, headloss is not a useful measurement. The head over the effluent weir however is an adequate measurement of filter effluent flowrate, which will decrease gradually throughout a filter run. When flowrate measurement is provided, head over the effluent weir would typically be measured by level probe or ultrasonic level sensor, calibrated for the weir characteristics to indicate flowrate. A low level (flow) set point would initiate an alarm, and a low-low level (flow) should shut down the filter. Alternatively, an in-line flowmeter may be used.

A continuous on-line turbidity monitor should be provided on each filter upstream of the filter-to-waste diversion or else a second turbidimeter should be provided for the filter-to-waste if the piping is separate. A high turbidity set point should alarm and/or initiate a backwash. A high-high set point should shut down the filter until the operator has investigated the cause of the high-high alarm and/or initiated a backwash.

Continuous recording of effluent turbidity is required in monitoring filter and plant performance.

C. Backwashing

Backwashing a filter, can be initiated various ways, as discussed earlier.

A time initiated backwash can be automatic. Smaller plants feeding smaller systems may benefit from backwashing overnight when demand is low - and the operator is not present. In such cases, a timer can be hard wired into the filter control panel to initiate the backwash, or alternatively, the time control can be programmed into the PLC.

If an automatic backwash on turbidity, headloss (constant rate) or flow (declining rate) is desired, care should be taken to consider plant demand and the effect of interrupting or reducing production to ensure that service and treatment are not compromised.

Before proceeding, the control system must confirm that a wash is permitted by checking source water volume, receiver volume, that no other filter is washing or that no other process or production restraints exist. Once started, its cycle can be controlled manually by the operator, automatically by timers and a sequencer in a PLC program. All timer settings must be adjustable.

The backwash cycle takes 20 to 40 minutes depending on whether it is a straight water backwash or includes surface wash or air scour. Backwash flow changes must be made gradually. Valve sequencing is important. Flow setpoints must be adjustable.

With surface wash, a typical cycle may include:

- draw down water level over filter
- initiate surface wash
- initiate backwash before surface wash ends
- filter-to-waste
- return filter to service

With air scour, the cycle may include:

- draw down water level over filter to approximately 50-100 mm above media
- initiate air scour
- combined air scour and low rate backwash on time or until level reaches within 50-75 mm of washwater waste weir
- stop air scour, initiate high rate backwash
- filter-to-waste until effluent turbidity is within limits
- return filter to service

A pressure transmitter senses water level over the filter to control the opening/closing of valves and the start/stop of surface wash, air scour and low rate backwash and filter filling after high rate backwash, if required. All other control is time based.

When a filter is put back into service, water should be filtered to waste until the turbidity has dropped to a preset acceptable value. Filter to waste should be performed at approximately the same flow rate as the filter normally operates. The on-line turbidimeter monitors filter-to-waste turbidity and once the turbidity set point is reached, a signal is sent for the filter-to-waste valve to close and the filter-to-production valve to open simultaneously. Valve stroking during this changeover should be synchronized to maintain flowrate through the filter as constant as possible to avoid the turbidity spikes that can occur with a sudden change in flow rate through a filter.

Water used for backwashing must be filtered water. A backwash pump will supply backwash water typically drawing from the plant's treated water clearwell. Alternatively, backwash water can be supplied directly off the transmission main leaving the plant or from a header tank. Backwash flow rate control is critical to assure good cleaning and avoid media loss. The higher density of cold water in winter requires a lesser flow rate than the summer to achieve fluidization of the bed. This flow rate set point should therefore be adjusted seasonally to compensate for the water temperature variation. Backwash flow rate is be measured by in-line (magnetic or ultrasonic) flowmeter.

Control of the flow rate may be manual or automatic. Manual control may be used on straight backwash systems (with no air scour). Manual control requires adjustment to a throttling valve on the backwash supply until the desired flow rate, as measured or the flowmeter, is achieved. This valve should be locked in this position and adjusted seasonally to compensate for water temperature as described above. Excessive and undesirable disturbance of the media can be caused by a sudden rush of water on pump start-up or rapid valve opening. This is avoided by having a second in-line valve - typically a pilot operated globe valve - that opens slowly to bring the backwash flow gradually to the set point. Alternatively, a bypass valve that closes slowly, may be used.

A backwash system that includes air scour - and consequently two backwash flow rates - is best controlled automatically by a PLC (or DCS). Backwash flow rate can be varied either by using variable speed drive on the backwash pump, using multiple pumps, or using an in-line flow control valve. For the variable speed drive and flow control valve options, a flow transmitter on the backwash supply will provide a signal to the PLC. The PLC will use this signal to control the pump speed/valve position.

For the multiple pump option, the pumps could be set up to provide the required flow rates using manually adjusted throttling valves on each pump discharge so that one pump could supply the low flow, and the other (or both in parallel) could meet high flow. As with manual control, flow rate changes should not be sudden, and an in-line pilot operated globe style valve should be used to avoid this.

b. Slow Sand Filtration

Although their function is the same as RGF, slow sand filters operate under different conditions. Slow sand filters (SSF) operate at much lower loading rates and rely on the formation of a thin layer - Schmutzdecke - on top of the sand medium. As well as forming a filter to remove particulates, this layer contains various micro-organisms that remove bacteria and other organisms. SSF are not backwashed; rather, the Schmutzdecke layer is removed occasionally (typically every 4-8 months) when headloss becomes excessive, flowrate drops off and/or quality starts to deteriorate.

Because of the very slow flow rate through SSF, headloss, flow rate and effluent quality can remain very stable for many weeks. Adjustments to the flow rate can be made manually by the operator.

When commissioned, SSFs have to mature over a period of several weeks before they can produce potable quality water. Once a filter has been brought on-line, the primary operator function is to monitor and control flow rate. Flow rate through a filter is measured either by measuring head over the effluent weir, or preferably by the more accurate method of in-line flow meter on the filter effluent pipe upstream of the filter recycle diversion. A manual throttling valve on the effluent pipe should be adjusted to ensure flow rate through a new filter is not excessive. A manual throttling valve on the filter inlet should also be adjusted to ensure influent flow rate closely matches the effluent flow rate. These valves should be adjusted to maintain the desired water depth over the bed. Alternatively, water level over the filter may be controlled by starting and stopping the raw water pumps. As the bed clogs and headloss increases, the effluent valve should be opened to compensate.

Instrumentation should be provided to routinely monitor raw and treated water quality. A sudden increase in headloss accompanied by a reduction in flow rate signals that the filter is plugged.

(vii) Disinfection

Chlorine is the most common disinfectant used in potable water treatment. The following discussion provides process control guidelines for the use of chlorine in either its gaseous or solution forms.

Effectiveness of chlorine disinfection and the dosage is a function of the water temperature, pH, the contact time and the chlorine demand. The dosage is controlled on the basis of the measured residual.

Disinfection is controlled:

a. to meet CT requirements, which will vary with the raw water quality and the performance of the upstream processes; and

b. to ensure that chlorine residual for water leaving the plant is maintained at the set point.

Laboratory chlorine demand analysis will establish the required dose concentration. The chlorine dose rate will then be controlled by a signal from the flow transmitter on the flowmeter measuring the flow in the process where the chlorine will be injected. The chlorine residual will be monitored downstream of the process/tankage that provides the required CT. This measured variable can be used to adjust automatically the dose concentration through feedback loop control, and/or provide an alarm status to which the operator would have to respond. Unless the plant is manned continuously, it is preferable that automatic control be provided.

Since treated water is stored at the plant for some time, its chlorine residual can diminish. The residual will thus need to be ‘topped up’ or trimmed to the proper concentration. The trim dose is controlled by the flowrate (not the plant production rate) and chlorine residual of the water leaving the plant. Feedback loop control using a chlorine residual analyzer combined with flow-pacing control from the discharge flowmeter will automatically maintain the set point residual. An alarm should be generated if the residual falls out of bounds.

Chlorine gas feed systems incorporate a low pressure switch on the vacuum gas line to the chlorine feeder to provide an alarm signal, and where applicable, signal an automatic switch over from duty chlorine cylinder/container to standby cylinder/container. Chlorine feed rate is controlled by modulating the chlorinator. The system is started and stopped by opening or closing the plant service water valve supplying the chlorine injector. A chlorine gas flow meter or solution flow proving switch is desirable. A chlorine gas detector must be provided in the chlorine room; the detector should be connected to a remote audible and visual alarm system.

6.3.5.8 Design Documents

Complete design documents should be prepared to ensure that construction can be completed correctly and also to properly record the system for future reference. The following are required in the design documents:

- Design and construction standards, specifications and installation details.
- Panel sizing and general arrangement.
- Control system functional requirements.
- Control component and instrument data sheets.
- Operator interface and control hardware and software specifications including input and output (I/O) lists.
- Control system programming and packaged system configuration standards, structure and scope.

6.3.5.9 Control System Documentation

The following documents should be provided following completion of the control system:

- Record drawings to show any changes to the design and including any drawings produced during construction.
- Annotated listings of control system programs and packaged system configuration.
- Manufacturer's literature for all control and instrumentation components.
- Final wiring diagrams complete with wire and terminal coding.
- Motor control schematics.
- Instrument loop diagrams.
- Panel wiring and layout details.
- PLC or DCS wiring schematics.
- Instrument calibration sheets.
- Operating instructions.

6.3.5.10 Training

Adequate training to the plant operating and maintenance staff should be provided so that the system can be operated to meet the design criteria.

Include safety training for chlorine and chemical handling, including spill clean-up and first aid.

6.3.6 Colour Codes for Water Treatment Plant Piping

TABLE 6.6

RECOMMENDED COLOUR CODING FOR WATER TREATMENT PLANT PIPING

Piping to be Identified	Basic Colour	Bands	
		No.	Colour
Raw or unfinished water	Dark Green	-	-
Clarified Water	Dark Green	1	Black
Filtered Water	Dark Green	2	Black
Filtered and Chlorinated (Potable) Water	Blue	1	-
Backwash Water	Light Green	-	-
Chemical Feed Lines	Pink	-	-
Coagulant	Pink	1	Black
pH Control	Pink	2	Black
Taste and Odour	Pink	3	Black
Fluoride	Pink	1	Green
Chlorine and Water	Pink	1	Yellow
Chlorine Gas	Yellow	-	-
Plumbing (Waste)	Brown	-	-
Electrical	Purple	-	-
Compressed Air	White	-	-
Heating	Silver	-	-
Fire Protection	Red	-	-
Natural Gas	Orange	-	-

Notes:
1. Entire length of pipe to be painted in basic colour.
2. Bands, if required, are to be placed as follows:
   (a) at 9 m intervals, and/or
   (b) where the pipe enters and leaves a room.
3. Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

6.3.7 Laboratory Requirements

All water treatment plants should have laboratory facilities for analytical testing as required in the terms and conditions of the operating approval. The capability of the laboratory should commensurate with the size and the complexity of water treatment operation. Before undertaking the detailed design of the laboratory facility, contact should be made with AEP to confirm the testing requirements.

Minimum laboratory requirements for surface water treatment plants are as follows:

1. Lab equipment

pH - pH meter
Chlorine - chlorine residual titrator or colorimetric test kit
Turbidity - turbidimeter
Colour - colour meter (spectrophotometer or colour comparator)

Tap aspirator/filter apparatus for colour samples
Glassware (beakers, measuring cylinders, sample bottles, flasks, pipettes, burettes)
Balance (top-loading) - if any chemical has to be weighed out
Clean lab bench area, with sink, adequate lighting, and storage

2. Lab chemicals/standards

pH butter solutions for calibration
Hydrochloric acid for cleaning pH probes and titrators
Distilled water or reagent grade water for rinsing
Turbidity standards
Colour standards
Reagents for any colorimetric tests
Lab grade dishwashing detergent

6.4 Water Transmission and Distribution Mains

6.4.1 Pipe Sizing

Watermains designed to carry fire flows should have a minimum inside diameter of 150 mm. For smaller distribution systems without fire flow provision, hydraulic calculations should verify that the proposed pipe sizes are sufficient to sustain the minimum operating pressure as defined in Section 6.1.4.

6.4.2 System Layout - Valves and Hydrants

6.4.2.1 Layout

Water distribution systems should be designed to eliminate dead-end sections. In cases where dead-end mains are unavoidable, measures should be taken to prevent stagnation.

It is advisable to have blow-off provision on all lines that have dead-ends and that are longer than 100 m.

6.4.2.2 Valve Placement

Water distribution systems should have shut-off valves located to allow any pipeline to be isolated for repairs.

Air release valves should be placed at all significant high points in the transmission system, and should also be considered at high points in the water distribution system. In addition, drain valves should be placed at low points of large mains to permit drainage during repairs to the distribution system.

6.4.2.3 Fire Flows and Hydrants

The provision of fire protection is solely the decision of the Local Authority.

Where hydrants are provided, the leads should be valved for easy maintenance. Where groundwater levels are above the hydrant drain port, the drains should be plugged and the barrels pumped dry for winter conditions.

For details regarding fire protection requirements in municipal waterworks system design, the designer should refer to the most current Fire Underwriters Survey publication entitled Water Supply for Public Fire Protection - A Guide to Recommended Practice.

6.4.3 Pumping

In general, the requirements for treated water pumping station are similar to those outlined in Section 6.2.3.3 for raw water pumping station.

The distribution system by pumping should be designed with at least two pumps. With one pump out of service, the remaining pumps should be able to deliver the maximum hourly design flow at not less than 150 kPa.

In order to supply water economically during low demand periods, at least one pump should be provided with a variable speed motor or an appropriately sized, small pump may be installed.

Standby power or an auxiliary gas powered pump should be provided to supply water during power outages or other emergencies. Fuel should be stored above ground and outside the water treatment plant building.

6.5 Potable Water Storage

6.5.1 Sizing

The total water storage requirements for a given water supply system where the treatment plant is only capable of satisfying the maximum daily design flow may be calculated using the following empirical formula:

S = (A + B + C) or D, whichever is greater

where
- S = Total storage requirement, m^3
- A = Fire storage, m^3
- B = Equalization storage (approximately 25% of projected maximum daily design flow), m^3
- C = Emergency storage (minimum of 15% of projected average daily design flow), m^3
- D = Disinfection contact time (T_{10}) storage to meet the CT requirements, m^3

The level of fire protection is the responsibility of the municipality. The level of storage may be further reduced if the water treatment plant is capable of supplying more than the maximum daily design flow or if there is sufficient flow data to support a lower peaking factor than would be normally used for the given population range.

The designer should recognize that the given formula for calculating treated water storage requirements must be supplemented with the storage required for the operation of the water treatment facility, i.e. filter backwash and domestic use.

6.5.2 Phasing

Treated water storage requirements should be calculated using a 'first phase' projected demand of no more than 10 years (refer to Section 6.1.2). Present worth cost analysis may show that longer design periods are more economical, however the failure to properly phase the storage requirements can result in operational problems due to oversized pumping facilities and/or problems with maintaining a required chlorine residual if the available storage is excessive.

6.5.3 Alternative Types

There are many alternative methods for the provision of treated water storage at either the water treatment plant or in the water distribution system. The choice of underground, ground level, or elevated storage will depend on factors such as the service area size, topography, and economics. Each alternative should be investigated to choose the best overall method of storage based on specific project conditions.

6.5.4 Site Selection

An economical site selection will depend on the type of reservoir, but in general the major factors to consider are soils conditions, compatibility with future expansion requirements, and site access.

In cases where it may be necessary to provide two-way flow direction during periods of high demand, it is recommended that the storage facility be located on the opposite site of the high demand centre from that at which the supply facilities discharge to the distribution system.

6.5.5 Design Considerations

6.5.5.1 Multi-Cell Provision

Underground or ground level storage reservoirs should be constructed with two cells for ease of maintenance. This can often be accomplished as a result of phasing requirements. Flexibility should be built in to operate the cells in series or parallel; or pump from either cell.
When planning the type of reservoir, the designer shall ensure that treated water is not stored or conveyed in a compartment adjacent to untreated water if the two compartments are separated by a single wall.

6.5.5.2 Access

Treated water storage reservoirs should have convenient access to the interior for cleaning and maintenance.

6.5.5.3 Vents

Reservoirs should be vented by specially designed vent structures. On ground level structures, vents should terminate in an inverted 'U' construction, the opening of which is at least 600 mm above the roof or sod, having a cover using suitable non-corrodible screen cloth.

6.5.5.4 Drains and Overflows

Where feasible, storage reservoirs should be provided with a drain. There shall be no direct connection between any storage reservoir drain or overflow and a storm or sanitary sewer. Overflows should be located at sufficient elevation to prevent the entrance of surface water. A backflow preventer should be installed on all overflows.

Freezing of overflow lines may result in rising water levels, frozen vents, and collapse of the reservoir by vacuum action. To prevent this occurrence, overflows should be designed with a large inlet weir to allow a substantial flow which is less likely to freeze. It is also desirable to have the overflow discharge to a warm environment.

6.5.5.5 Circulation

It is most desirable to have perfect plug flow through a reservoir. Such a design would ensure that all water entering the reservoir had the same detention time. This is particularly important in the post treatment reservoir to ensure as high a T_{10} value as possible. Baffling design and inlet and outlet design can lead to a near plug flow operation. Refer to Appendix C for illustration of typical baffling conditions.

6.6 Watering Point

Watering point can be either a truck fill station or a barrel fill station.

The truck fill supply should have a minimum pumping capacity of 1000 L/min. to minimize the truck fill time; a separate pump should be provided for barrel fill supply. All water supplied from the facility should be metered.

An exterior overhead truck fill arm should be installed. The design shall be such that there will be no cross contamination during or after filling the truck or barrel.

An exterior coin meter activated pump control should be installed so that no person needs to enter the building. The panel should include start and stop buttons, a pump selector switch, and an adjustable maximum run timer or volume control.

6.7 Security and Protection

6.7.1 Excavations and Open Trenches

In order to ensure public safety, Local Authorities responsible for the construction shall secure excavations and open trenches during non-working periods by installing fences/barricades and/or warning lights/signs.

6.7.2 Reservoirs

To guard against trespassing and vandalism, above ground treated water storage reservoirs should be fenced, and entrance gate, access manholes, and valves or vent houses should be locked. The reservoirs should also have water-tight roofs, and any openings should have suitable covers to prevent the entrance of birds, animals, insects or dust.

Where a raw water reservoir is located near developed areas or agricultural land, fencing is required to discourage attempts by children to gain entry and to preclude the entrance of domestic animals. Fencing should be located outside the toe of the berm; chain link fence is recommended. Fishing will not be permitted in raw water reservoirs.

7.0 DESIGN GUIDELINES - WASTEWATER SYSTEM

7.1 Design Criteria

7.1.1 Estimating Wastewater Flows

The following sections outline methodologies for quantifying wastewater flows. From a qualitative point of view, owners of wastewater systems are encouraged to develop and implement policies and programs to promote "at source reduction" for any and all contaminants in wastewater.

7.1.1.1 Residential (Population-Generated)

If no existing data exists, the peak (population-generated) flow for a residential population may be determined by the following formula:

$$Q_{PDW} = \frac{G \times P \times Pf}{86.4}$$

where: Q_{PDW} = the peak dry weather design flow rate (L/s)
G = the per capita average daily design flow (L/d)
P = the design contributing population in thousands
Pf = a "peaking factor".

The peaking factor (Pf) should be the larger of 2.5 or Harmon's Peaking Factor

where:

Harmon's Peaking Factor = $1 + 14/(4 + P^{1/2})$

where:
P = the design contributing population in thousands

7.1.1.2 Commercial/Institutional and Industrial

1. Determination of Average Flow

For detailed system design, the average wastewater flow from commercial/ institutional and industrial land use areas is to be estimated as set out in Table 7.1 or by actual documented usage.

TABLE 7.1
EXPECTED VOLUME OF SEWAGE PER DAY*

Place	Estimated Sewage Flow Litres (gallons) Per Day
Assembly Halls	32 (7) per seat
Campsite	80 (18) per campsite
Churches	23 (5) per seat
with kitchen	32 (7) per seat
Construction Camps	225 (50) per person
Day Care Centre	113 (25) per child
Dwellings	675 (150) per bedroom
Golf Clubs	45 (10) per member
with bar and restaurant add	113 (25) per seat
Hospital	
(no resident personnel)	900 (200) per bed
Industrial and Commercial Buildings	
(does not include process water or cafeteria)	45 (10) per employee
(with showers)	90 (20) per employee
Institutions	
(resident)	450 (100) per resident
Laundries	
(coin operated)	1800 (400) per machine
Liquor Licence Establishments	113 (25) per seat
Mobile Home Parks	1350 (300) per space
Motels/Hotels	90 (20) per single bed
Nursing and Rest Homes	450 (100) per resident
Office Buildings	90 (20) per employee
Recreational Vehicle Park	180 (40) per space
Restaurants	
24-Hour	225 (50) per seat
Not 24-Hour	160 (35) per seat
Schools	
Elementary	70 (15) per student
Junior High	70 (15) per student
High School	90 (20) per student
Boarding	290 (65) per student
Service Stations	
(exclusive of cafe)	560 (125) per fuel outlet
Swimming Pools (Public)	
based on design bathing load	23 (5) per person

***** Reproduced from the Alberta Private Sewage Treatment and Disposal Regulations, Table 8.5.B.

2. Average Flow Generation Estimates for Planning

For system planning purposes, when specific land uses and zoning are unknown and the requirements of 7.1.1.2 (1) cannot be defined, the recommended lower limits for estimation of average flow generation (to be used for preliminary planning unless the use of other values is justified with more specific or reliable information) are as follows:

(i) Commercial and Institutional Land Uses

The lower limit for Average Flow Generation should be 40 m^3/day/ha (0.46 L/s/ha).

(ii) Industrial Land Uses

The lower limit for average flow generation should be 30 m^3/day/ha (0.35 L/s/ha).

3. Determination of Peak Dry Weather Flow Rate

Peak dry weather flow rates for specific design areas are to be determined by application of a peaking factor (Pf), related to the average flow rate (Q_{AVG} in L/s) in accordance with the following expression to a maximum value of 5.0:

$$Pf = 6.659 \left(Q_{AVG}^{-0.168}\right)$$

Following from this, the peak dry weather flow rate (Q_{PDW} in L/s) may be determined as follows:

$$Q_{PDW} = Pf.Q_{AVG}$$
$$= 6.659 \left(Q_{AVG}^{0.832}\right)$$

4. Special Considerations - High-Water-Consumption Land Uses

The foregoing guidelines may not be applied to high water consumption land uses such as heavy industry, meat packing plants, breweries, etc. Detailed analysis of the design requirements specific to each development proposal is required in such cases.

5. Residential Components of Commercial Developments

Where proposed commercial developments include discretionary residential components, the sanitary flow generation from the residential component should be determined in accordance with Section 7.1.1.1, and is to be included in the determination of the total generation for the development.

7.1.1.3 Extraneous Flow Allowance - All Land Uses

In computing the total peak flow rates for design of sanitary sewers, the designer should include allowances as specified below to account for flow from extraneous sources.

1. **General Inflow/Infiltration Allowance**

A general allowance of 0.28 L/s/ha should be applied, irrespective of land use classification, to account for wet-weather inflow to manholes not located in street sags and for infiltration flow into pipes and manholes.

In addition, a separate allowance for inflow to manholes located in street sags should be added as per the next section.

2. **Inflow Allowance - Manholes in Sag Locations**

When sanitary sewer manholes are located within roadway sags or other low areas, and are thus subject to inundation during major rainfall events, the sanitary design peak flow rate should be increased by 0.4 L/s for each such manhole, which is applicable for manholes which have been waterproofed. For new construction, all sanitary manholes in sag locations are to be waterproofed.

For planning purposes and downstream system design, where specific requirements for an area are unknown, the designer should make a conservative estimate of the number of such manholes which may be installed in the contributing area based on the nature of the anticipated development, and include an appropriate allowance in the design.

3. **Others**

In areas where weeping tiles are connected to the sanitary sewer system, an additional amount, based on on-site measurements, should be included in the design flow. The designer should also take into account the pipe material and soil type in determining the extraneous flow allowance.

7.1.1.4 Total Peak Design Flow Rates

The total peak design flow rates should be the sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area.

7.1.2 System Capacity

In general, sewer capacities should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. For example, the wastewater treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 or 25 year design flow. Outfall structures, which have high base construction costs, should be designed for the entire design horizon which is usually about 20 to 25 years. The decision is best made based on economic analysis and cost return.

7.1.3 Wastewater Collection and Treatment System

Wastewater collection system including the pumping stations should be designed for peak wastewater design flows.

Aerated lagoon systems should be designed for maximum monthly average daily design flows, with sufficient aeration to maintain a uniform solids concentration in the complete mix cell.

Mechanical wastewater treatment plants should be hydraulically capable of handling the anticipated peak wastewater design flow rates without overtopping channels and/or tankage. From a process point-of-view, however, the design of various components of the plant should be based on the following:

Screening/Grit Removal - Peak wastewater design flow rate.

Primary Sedimentation - Average design flow rates or peak wastewater design flow rate.

Aeration - Maximum monthly average CBOD loading rate in the design year is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Seasonal variations in domestic and/or industrial CBOD loading rates should also be taken into consideration. Except for short retention treatment systems such as contact stabilization or high rate processes, hydraulic retention time is seldom critical.

Secondary Sedimentation - Peak wastewater design flow rates or peak solids loading rate.

Disinfection System - Peak wastewater design flow rates.

Effluent Filtration - Peak wastewater design flow rates.

7.1.4 Sewer Outfall

Sewer outfall should be designed for peak wastewater design flow rates.

The proper siting and design of the sewer outfall is important in minimizing the impact on receiving water quality. Outfalls should be designed and located so as to obtain the greatest possible dilution of the effluent as quickly as possible during low flow periods.

Dilution is a product of initial mixing of the effluent with surrounding water and subsequent dispersion due to water movement. Initial mixing is enhanced by extending the outfall away from the shore into deeper water and often by incorporating a multiport diffuser to spread the discharge over a larger area and to increase turbulent mixing. Similarly, dispersion is aided by maximizing the separation of the discharged plume from boundary effects of the shoreline or streambed.

7.2 Wastewater Collection

7.2.1 Sewers

7.2.1.1 Materials

The material selected should be adapted to local conditions, such as: character of wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, hydrogen sulphide corrosion, and similar problems.

Suitable couplings shall be used for joining dissimilar materials.

All sewers should be designed to prevent damage from superimposed live, dead, and frost induced loads. Proper allowance for loads on the sewer should be made because of soil and potential groundwater conditions, as well as the width and depth of trench. Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction should be used to withstand anticipated potential superimposed loading or loss of trench wall stability.

For application in which the wastewater is conveyed under pressure, or in special cases involving excessive surcharge such as inverted siphons, pressure pipes should be used. Pipe and joints should be equal to watermain strength materials suitable for design conditions.

7.2.1.2 Sizing of Sewers

It is normal practice to design sanitary sewers to have a hydraulic capacity such that the sewer is flowing at no more than 80% of the depth when conveying the estimated design peak flow. This is because the maximum velocity is achieved when the flow is at about 0.8 of depth (note: maximum flow occurs when the pipe is flowing at about 0.93 of depth. The reason for this is that as a section approaches full flow, the additional friction resistance caused by the crown of the pipe has a greater effect than the added cross sectional area).

Flow rate at a depth of 80% of the sewer diameter is approximately 86% of the sewer full capacity. Therefore, the required flow capacity for sizing of the sewer is computed using the following relationship:

$$\textit{Required sewer capacity} = \frac{\textit{Estimated design flow}}{0.86}$$

Manning equation is generally used in sizing the sewers:

$$Q = \frac{1.00}{n} AR^{2/3} S^{1/2}$$

where: Q = Quantity of flow (m^3/s)
n = Roughness coefficient (common value used is 0.013; lower value may be used for PVC pipes based on manufacture's recommendation)

A	=	Cross sectional area of flow (m^2)
R	=	Hydraulic Radius (m)
S	=	Slope (m/m)

7.2.1.3 Changes in Pipe Size

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

7.2.1.4 Location

At the discretion of the municipality, the sewers may be located on the sides of the undeveloped road allowances or on the verges of developed roads.

7.2.1.5 Pressure Testing

Testing of sewers is recommended when high water table is expected or encountered.

The infiltration/exfiltration rate for PVC sewer pipes and fittings may not exceed 4.6 litres per mm diameter of pipe per km length per day. Low pressure air-testing may be permitted to verify this joint tightness when tested to a maximum rate of air loss of 0.0015 ft^3 per minute per ft^2 of internal surface. Test methods to the requirements of Uni-Bell Standard UNI-B-6-90; See Appendix E for test time calculation.

7.2.2 Manholes

Manholes should be durable structures for the purpose of providing convenient access to sewers for observations, inspections, flow monitoring and maintenance operations, at the same time causing a minimum of interference in the hydraulics of the sewer system.

Where corrosion is expected because of either sulphate attack or sulphide, consideration shall be given to the provision of corrosion resistant material or effective protective linings.

7.2.2.1 Location

Manholes should be installed:

- at the end of each line;
- at all changes in grade, size, or alignment;
- at all intersections; and at distances not greater than 120 m for sewers 375 mm or less; or
- 150 m for sewers 450 mm to 750 mm.

However, the limits may be exceeded if suitable modern cleaning equipment is available to handle the larger spacing.

Greater spacing may be allowed in sewers larger than 750 mm.

Cleanouts may be used only for special conditions and may not be substituted for manholes nor installed at the end of laterals greater than 50 m in length.

Manholes should not be located in areas subject to ponding during rainstorms and snow melt.

7.2.2.2 Sizing

For sewers up to 1050 mm in size, manholes should be constructed with a diameter of at least 1200 mm. For sewers larger than 1050 mm, special type manholes or tee riser manholes may be used. Safety and entry requirements should also be considered when sizing manholes.

7.2.2.3 Drop Manholes

Drop manholes should be used when invert levels of inlet and outlet sewers differ by 600 mm or more. Where the difference in elevation is less than 600 mm, the 0.8 depth point of both sewers should be matched.

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

Due to the unequal earth pressures that would result from the backfilling operation in the vicinity of the manhole, the entire outside drop connection shall be encased in concrete.

7.2.2.4 Channelling and Benching

Good design practice should prevent the depth of flow from being above the side walls of the manhole channelling at all times. Therefore, manhole channels should be a smooth continuation of the incoming pipe, the channel height being one-half the pipe diameter for small sewers or three-fourths the pipe diameter for large sewers (375 mm or larger).

Manhole benching should ensure both good footing for workmen and adequate space for minor tools and equipment. Benching should have enough slope for drainage, however to provide safe footing the slope should not exceed 80 mm/m.

No lateral sewer, service connection, or drop manhole pipe should discharge onto the surface of the benching.

7.2.2.5 Frame and Cover

Manhole covers should be designed having the following:

1. Adequate strength to support superimposed loads. Frames and covers are usually cast iron, however lighter weight materials may be used where there is no danger of subjection to heavy loads;

2. Adequate size to facilitate access of equipment and people;

3. A good fit between cover and frame to prevent rattling in traffic;

4. Water tightness between cover and frame to reduce infiltration.

5. Provision for ease of opening (usually a pick notch to pry the cover loose) and an additional pick hole near the edge of the cover;

6. Provision of vent holes; and

7. Resistance to unauthorized entry. The principle defence against a manhole cover being lifted by children is its weight, however during infrequent storm events it is possible that surcharge and lifting of the cover can occur. Therefore, provision should be made in the design to eliminate the possibility of a person falling into the manhole if the cover has been dislodged.

7.2.2.6 Steps

Manhole steps should be either aluminum or galvanized steel, being wide enough to place both feet on one step. Spacing of steps should be 300 to 400 mm.

To reduce the possibility of feet slipping on manhole steps, the safety-drop type of steps are recommended. For those manholes located within a road-way, and where possible, steps should be aligned so that the person exiting from the manhole should do so facing towards oncoming traffic.

7.2.3 Inverted Siphons

Inverted siphons should have not less than two barrels, with a minimum pipe size of 100 mm. They should be provided with necessary appurtenances for maintenance, convenient flushing, and cleaning equipment. The inlet and discharge structures should have adequate clearances for cleaning equipment, inspection, and flushing. Design should provide sufficient head and appropriate pipe sizes to secure velocities of at least 1 m/s for average design flows. The inlet and outlet details should be so arranged that the flow is diverted to one barrel, and so that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

7.2.4 Wastewater Pump Station

7.2.4.1 General

Wastewater pump stations in general use fall into four types:

1. wet well/dry well;
2. submersible;
3. suction lift; and
4. screw pump.

Once the need for a pump station has been determined, the designer should select the type and location that offers a proper balance between the technical needs, economics, and the environment.

Special consideration should be given to the location of the structure relative to neighbouring development in order to minimize the possible effects of noise and odour.

All weather vehicular access should be provided to all pump stations. Security fencing and access hatches with locks should also be provided.

7.2.4.2 Wet Well/Dry Well Pump Station

1. Structures

Safety ventilation, well separation, access and safety requirements shall be in accordance with the details outlined in Section 5.1.2.

(i) Equipment Removal

Provision should be made to facilitate removing pumps, monitors, and other mechanical and electrical equipment.

(ii) Buoyancy

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

2. Pumps

(i) Protection Against Clogging

Pumps handling wastewater from 750 mm or larger diameter sewers should be preceded by readily accessible bar racks to protect the pumps from clogging or damage. Bar racks should have clear openings as provided in Section 7.3.1.4. Where a bar rack is provided, a mechanical hoist should also be provided.

(ii) Pump Openings

Pumps handling raw wastewater should be capable of passing particles of at least 75 mm in diameter. Pump suction and discharge openings should be at least 100 mm in diameter.

(iii) Priming

The pump should be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 7.2.4.3.

(iv) Electrical Equipment

Electrical systems and components (e.g. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in raw wastewater wet wells, or in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapours may be present, should comply with the Canadian Electrical Code requirements for Class I Group D, Division 1 locations. In addition, equipment located in the wet well should be suitable for use under corrosive conditions. Each flexible cable should be provided with a watertight seal and separate strain relief. A fused disconnect switch located above ground should be provided for the main power feed for all pumping stations. When such equipment is exposed to weather, it should be weather proofed. A 110 volt power receptacle to facilitate maintenance should be provided inside the control panel for lift stations that have control panels outdoors. Ground fault interruption protection should be provided for all outdoor outlets.

(v) Intake

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

(vi) Dry Well Dewatering

A sump pump equipped with dual check valves should be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal leakage shall be piped or channelled directly to the sump. The sump pump should be sized to remove the maximum pump seal water discharge which would occur in the event of a pump seal failure.

(vii) Pumping Rates

The pumps and controls of main pumping stations should be selected to operate at varying delivery rates. Insofar as is practicable, such stations should be designed to deliver as uniform a flow as practicable in order to minimize hydraulic surges. The design flow should be adequate to maintain a minimum velocity of 0.6 m/s in the forcemain.

3. Valves

(i) Suction Line

Suitable shut-off valves should be placed on the suction line of dry pit pumps.

(ii) Discharge Line

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

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

4. Wet Wells

(i) Divided Wells

Where continuity of pumping station operation is critical, consideration should be given to dividing the wet well into two sections, properly interconnected, to facilitate repairs and cleaning.

(ii) Size

The design fill time and minimum pump cycle time should be considered in sizing the wet well. The effective volume of the wet well should be based on design average flow and a filling time not to exceed 30 minutes unless the facility is designed to provide flow equalization/storage. The pump manufacturer's duty cycle recommendations may be utilized in selecting the minimum cycle time. When the anticipated initial flow to the pumping station is less than the design average flow, provisions should be made so that the fill time indicated is not exceeded for initial flows. When the wet well is designed for flow equalization as part of a treatment plant, provisions should be made to prevent septicity. The well and the pumps should also be configured to avoid settlement of solids in the wet well.

(iii) Floor Slope

The wet well floor should have a minimum slope of 1 to 1 to the hopper bottom. The horizontal area of the hopper bottom should be no greater than necessary for proper installation and function of the inlet.

(iv) Air Displacement

Covered wet wells should have provisions for air displacement such as an inverted "J" tube or other means which vents to the outside.

5. Flow Measurement

Suitable devices for measuring wastewater flow should be provided at all pumping stations. Indicating, totalizing and recording flow measurement devices/instruments should be provided at large pumping stations with peak design flow greater than 50 L/s. Elapsed time meters may be used for pump stations with peak design flow less than 50 L/s.

7.2.4.3 Suction-Lift Pump Station

Pump Priming and Lift Requirements

Suction-lift pumps should be of the self-priming type and should meet the applicable requirements of Section 7.2.4.2. Suction-lift pump stations using dynamic suction lifts may exceed the limits outlined in the following sections if the manufacturer certifies pump performance and submits detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations should include static suction-lift as measured from "lead pump off" elevation to centre line of pump suction, friction, and other hydraulic losses of the suction piping, vapour pressure of the liquid, altitude correction, required net positive suction head, and a safety factor of at least 1.8 m.

The pump equipment compartment should be above grade or offset and shall be effectively isolated from the wet well to prevent the humid and corrosive sewer atmosphere from entering the equipment compartment.

1. Self-Priming Pumps

Self-priming pumps should be capable of rapid priming and repriming at the "lead pump on" elevation. Such self-priming and repriming should be accomplished automatically under design operating conditions. Suction piping should not exceed 7.6 m in total length. Priming lift at the "lead pump on" elevation should include a safety factor of at least 1.2 m from the maximum allowable priming lift for the specific equipment at design operating conditions. The combined total of dynamic suction-lift at the "pump off" elevation and required net positive suction head at design operating conditions should not exceed 6.7 m.

2. Vacuum-Priming Pumps

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

7.2.4.4 Submersible Pump Stations

Submersible pump stations should meet the applicable requirements under Section 7.2.4.2, except as modified in this Section.

1. Construction

Submersible pumps and motors should be designed specifically for raw wastewater use, including totally submerged operation during a portion of each pumping cycle and should meet the requirements of the Canadian Electrical Code for such units. An effective method to detect shaft seal failure or potential seal failure should be provided.

2. Pump Removal

Submersible pumps should be readily removable and replaceable without dewatering the wet well or disconnecting any piping in the wet well.

3. Electrical

(i) Power Supply and Control

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

(ii) Controls

The motor control centre should be located outside the wet well, be readily accessible, and be protected by a conduit seal or other appropriate measures meeting the requirements of the Canadian Electrical Code, to prevent the atmosphere of the wet well from gaining access to the control centre. The seal should be so located that the motor may be removed and electrically disconnected without disturbing the seal.

(iii) Power Cord

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

4. Valves

Valves required under Section 7.2.4.2 (4) should be located in a separate valve pit. Valve pits may be dewatered to the wet well through a valved drain line. Check valves that are integral to the pump need not be located in a separate valve pit provided that the valve can be removed from the wet well in accordance with Section 7.2.4.4 (2).

7.2.4.5 Alarm Systems

Alarm systems should be provided for pump stations. The alarm should be activated in cases of high and low liquid levels, power failure, sump pump failure, pump failure, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms should be telemetered to a municipal facility that is manned 24 hours a day. If such a facility is not available and a 24-hour holding capacity is not provided, the alarm should be telemetered to municipal offices during normal working hours and to the home of the responsible person(s) in charge of the lift station during off-duty hours. Audio-visual alarm systems with a self-contained power supply may be installed in some cases in lieu of the telemetering system outlined above, depending upon location, station holding capacity and inspection frequency.

7.2.4.6 Emergency Operation

1. Objective

Wastewater pumping stations should be designed and operated in such a way that equipment breakdown may not result in the discharge of raw or partially treated wastewater to any waters and to protect public health by preventing back-up of wastewater and subsequent discharge to basements, streets, and other public and private property.

2. Emergency Pumping Capability

Emergency pumping capability should be included unless on-system overflow prevention is provided by adequate storage capacity. Emergency pumping capability may be accomplished by connection of the station to at least two independent power grids, or by provision of portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy, or by the provision of portable pumping equipment. Such emergency standby systems should have sufficient capacity to start up and maintain the total related running capacity of the station. Regardless of the type of emergency standby system provided, a riser from the forcemain with rapid connection capabilities and appropriate valving should be provided for lift stations to hook up portable pumps.

3. Emergency High Level Overflows

For use during possible periods of extensive power outages, mandatory power reductions, or uncontrollable emergency conditions, consideration should be given to providing a controlled, high-level wet well overflow to supplement alarm systems and emergency power generation in order to prevent backup of wastewater into basements, or other discharges which may cause severe adverse impacts on public interests, including public health and property damage. Where a high level overflow is utilized, consideration should also be given to the installation of storage/detention tanks, or basins, which should be made to drain to the station wet well. Overflows should be considered only in conjunction with emergency pumping capability as outlined in Section 7.2.4.6 (2).

4. Equipment Requirements

(i) General

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

(a) Engine Protection

The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment should be capable of shutting down the engine and activating an alarm on site and as provided in Section 7.2.4.5. Protective equipment should monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

(b) Size

The engine should have adequate rated power to start and continuously operate under all connected loads.

(c) Fuel Type

Reliability, and ease of starting, especially during cold weather conditions, should be considered in the selection of the type of fuel.

(d) Engine Ventilation

The engine should be located above grade with adequate ventilation of fuel vapours and exhaust gases.

(e) Routine Start-Up

All emergency equipment should be provided with instructions indicating the need for regular starting and running of such units at full loads.

(f) Protection of Equipment

Emergency equipment should be protected from damage at the restoration of regular electrical power.

(ii) Engine-Driven Pumping Equipment

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

(a) Pumping Capacity

Engine-driven pumps should meet the design pumping requirements unless storage capacity is available for flows in excess of pump capacity. Pumps should be designed for anticipated operating conditions, including suction lift if applicable.

(b) Operation

The engine and pump should be equipped to provide automatic start-up and operation of pumping equipment unless manual start-up and operation is justified. Provisions should also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm system must meet the requirements of Section 7.2.4.6 (4) (ii) (c).

(c) Portable Pumping Equipment

Where part or all the engine-driven pumping equipment is portable, sufficient storage capacity with alarm system should be provided to allow time for detection of pump station failure and transportation and hook-up of the portable equipment.

(iii) Engine-Driven Generating Equipment

Where permanently-installed or portable engine-driven generating equipment is used, the following requirements should apply in addition to general requirements:

(a) Generating Capacity

- Generating unit size should be adequate to provide power for pump motor starting current and for lighting, ventilation, and other auxiliary equipment necessary for safety and proper operation of the lift station.

- Special sequencing controls should be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

(b) Operation

Provisions should be made for automatic and manual start-up and load transfer unless only manual start-up and operation is justified. The generator should be protected from operating conditions that would result in damage to equipment. Provisions should be considered to allow the engine to start and stabilize at operating speed before assuming the load. Where manual start-up and transfer is justified, storage capacity and alarm system should meet the requirements of Section 7.2.4.6 (4) (iii) (c).

(c) Portable Generating Equipment

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

7.2.5 Forcemains

7.2.5.1 Velocity and Diameter

At design pumping rates, a cleansing velocity of at least 0.6 m/s should be maintained.

7.2.5.2 Air and Vacuum Relief Valve

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

7.2.5.3 Termination

Forcemains should enter the gravity sewer system at a point not more than 600 mm above the flow line of the receiving manhole.

7.2.5.4 Design Pressure

The forcemain and station piping should be designed to withstand water hammer pressures and associated cyclic reversal of stresses that are expected with the cycling of wastewater lift stations. Surge protection systems should be evaluated.

7.2.6 Security of Open Trenches and Excavations

In order to ensure public safety, Local Authorities responsible for the construction should secure open trenches and excavations during non-working periods by installing fences/barricades and/or warning lights/signs.

7.3 Wastewater Treatment

7.3.1 Mechanical Wastewater Treatment

7.3.1.1 Site Selection

1. Plant Location

Some of the factors which should be taken into consideration when selecting a new plant site are as follows:

(i) Setback distances from land use surrounding plant site [see Section 5.2.3.2 (1)];

(ii) Susceptibility of site to flooding [See Section 5.2.1.3 (2)];

(iii) Prevailing wind direction; and

(iv) Adequacy of site for future expansion.

2. Plant Layout

Plant buildings should be situated to provide adequate allowances for future expansions of the various treatment sections. The plant should also be oriented so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, noise, misting, freezing problems, energy consumption, and other environmental impacts. The plant layout should also allow for the probability of snow drifting. Entrances, roadways and open tankage should be located so that the effect of snow drifting on operations will be minimized.

Processing units should be arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludges, or chemicals from one module to the next, and also to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

Vehicular access should be sufficient to allow for the largest anticipated delivery or disposal, with allowance made to accommodate vehicle turning and forward exit from the plant site.

3. Provision for Expansion

In addition to the general site considerations, there are a number of allowances needed to provide for economical and practical expansion of the wastewater treatment facilities. Key provisions include:

(i) Design of on-site pumping stations such that their capacity can be increased and/or parallel facilities constructed without the need for major disruption of the plant's operation;

(ii) Layout and sizing of channels and plant piping such that additional treatment units can be added or increases in loading rates accommodated. Similarly, the layout of buildings and tankage should accommodate the location of the future stages of expansion;

(iii) Space provision within buildings to provide for replacement of equipment with larger capacity units. This is particularly important with equipment such as pumps, blowers, boilers, heat exchanges, etc. Adequate working space should be provided around equipment, and provision made for the removal of equipment; and,

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

7.3.1.2 Plant Hydraulics

1. Wastewater Pumpage

Raw wastewater and any intermediate wastewater pump stations associated with wastewater treatment works should be capable of conveying the peak wastewater flow rates to downstream treatment units. Pumping equipment should also be designed so that downstream treatment units are not subjected to unnecessary surging. This is best achieved by providing variable capacity, or multiple fixed capacity pumps, so that pump discharge rates will closely match the sewage inflow rate. [See also Section 7.2.4.2 (2) (vii)].

2. Channel Flow

Channels should be designed to convey the initial and ultimate range of flows expected. To avoid solids build-up, the following scouring velocities should be developed in normally used channels at least once per day:

Wastewater containing grit	- 0.9 m/s
Wastewater containing floc suspensions	- 0.45 to 0.60 m/s

Where the above scouring velocities cannot be obtained, channels may be aerated to prevent solids deposition.

3. Flow Division

Within wastewater treatment plants, there will invariably be situations where flow splitting is necessary. Unless certain precautions are taken, the flow will not split in the proportions desired over the full flow range, or the flow may split properly, but the organic load will not be divided in the same proportion.

To ensure that the organic load splits in the same proportion as the flows, the suspended solids should be homogeneously dispersed throughout the liquid and the relative momentum of all particles should be approximately equal at the point of diversion. Some turbulence is therefore desirable before each point of diversion. The following methods can be used to produce homogeneity:

- mechanical mixers;
- diffused aeration;
- bottom entrance into splitting box; or
- bar racks or posts in channels.

4. Plant Hydraulic Gradient

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

(i) head losses due to channel and pipe wall friction;

(ii) head losses due to sudden enlargement or sudden contraction in flow cross section;

(iii) head losses due to sudden changes in direction, such as at bends, elbows, Y-branches and tees;

(iv) head losses due to sudden changes in slope, or drops;

(v) head losses due to obstructions in conduit;

(vi) head required to allow flow over weirs, through flumes, orifices and other measuring, controlling, or flow division devices;

(vii) head losses caused by flow through comminutors, bar screens, tankage, filters and other treatment units;

(viii) head losses caused by air entrainment or air binding;

(ix) head losses incurred due to flow splitting along the side of a channel;

(x) head increases caused by pumping; and

(xi) head allowances for expansion requirements and/or process changes.

Consequences of excessive or inadequate allowances for head losses through wastewater treatment works should be noted. If pumpage is required, excessive head loss allowances result in energy wastage. If inadequate head loss allowances are made, operation will be difficult and plant expansion more costly.

7.3.1.3 Wastewater Characterization

Detailed wastewater characterization studies should be undertaken whenever existing data is limited or of suspect quality. Industrial discharges to municipal systems can significantly alter the characteristics and treatability of domestic wastewater. Wastewater containing industrial discharges should therefore be thoroughly characterized before selecting and designing biological treatment process units. Ideally, the undiluted industrial wastewater itself should be characterized so that any spikes originating from that source can be accounted for in the design of the process units. Oil and grease, pH, phosphorus and nitrogen levels should also be determined prior to selection and design of the treatment process.

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

Biological treatment process units are generally designed using total BOD loadings, however, in some cases soluble BOD loadings may be used where recommended by equipment suppliers. In such cases it is generally assumed that soluble BOD represents a certain fraction of the total BOD. Because wastewater characteristics can vary significantly, the actual ratio of soluble to total BOD should be determined whenever soluble BOD is used for design purposes.

Optimum growth of the micro-organisms is dependent on the supply of essential nutrients and trace elements. In addition to carbon, the two most critical elements are nitrogen and phosphorus. To encourage the growth of the organism, it is advisable to maintain a BOD:N:P ratio of 100:5:1. Failure to maintain a balanced nutrient level could result in operational problem. If necessary, nutrients may have to be added to the wastewater to provide a balanced level for microbial growth.

While the design of the plant to treat the domestic component of the total BOD may be straightforward, the potential difficulties with biological stabilization of the industrial wastewater flows should be recognized. Therefore, the industrial flow component should be characterized by using the following ratio:

$$\frac{\text{BOD of the industrial flow at 5 days and } 20^0\text{C}}{\text{BOD of the industrial flow at 20 days and } 20^0\text{C}}$$

Many industrial wastes also contain substances that may exert toxic effects on the organisms. Phenol, cyanide, ammonia, sulphide, heavy metals and many organic compounds may completely inhibit the microbial activity if these concentrations exceed the limit which an be tolerated by the micro-organisms. Thus, waste characterization is extremely important in the design of the plant.

7.3.1.4 Preliminary Treatment

Preliminary treatment consists of screening and grit removal.

Screening is provided as the first treatment stage for the protection of plant equipment against blockage, reduced operating efficiency, or physical damage.

Grit removal is required to prevent the undue wear of machinery and unwanted accumulation of solids in channels, settling tanks and digesters.

1. Screening Devices

(i) Coarse Screens

(a) Where Required

Protection for pumps and other equipment should be provided by trash racks, coarse bar racks, or coarse screens.

(b) Design and Installation

- *Bar Spacing*

  Clear openings between bars should be no less than 25 mm for manually cleaned screens. Clear openings for mechanically cleaned screens may be greater than 6 mm. Maximum clear openings should be 50 mm.

- *Slope and Velocity*

  Manually cleaned screens should be placed on a slope of 30 to 45 degrees from the horizontal.

  Approach velocities should be no less than 0.5 m/s to prevent settling; and no greater than 1 m/s to prevent forcing material through the openings, during normal variations inflow conditions.

- *Channels*

  Dual channels should be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions should also be made to facilitate dewatering each unit. The channel preceding and following the screen should be shaped to eliminate standing and settling of solids.

- *Auxiliary Screens*

  Where a single mechanically cleaned screen is used, an auxiliary manually cleaned screen should be provided. Where two or more mechanically cleaned screens are used, the design should provide for taking any unit out of service without sacrificing the capability to handle the peak design flows.

- *Invert*

  The screen channel invert should be 75-150 mm below the invert of the incoming sewer.

- *Flow Distribution*

  Entrance channels should be designed to provide equal and uniform distribution of flow to the screens.

- *Backwater Effect on Flow Metering*

  Flow measurement devices should be selected for reliability and accuracy. The effect of changes in backwater elevation, due to intermittent cleaning of screens, should be considered in locations of flow measurement equipment.

- *Freeze Protection*

  Screening devices and screening storage areas should be protected from freezing.

- *Screenings Removal and Disposal*

  A convenient and adequate means for removing screenings should be provided. Hoisting or lifting equipment may be necessary depending on the depth of pit and amount of screenings or equipment to be lifted.

  Facilities should be provided for handling, storage, and disposal of screenings. Screenings should be disposed of at the sanitary landfill.

  Manually cleaned screening facilities should include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities should be provided for both the platform and the storage area.

(ii) Fine Screens

(a) General

Fine screens should have openings of approximately 1.5 mm. The amount of material removed by fine screens is dependent on the waste stream being treated and screen opening size.

Fine screens should not be considered equivalent to primary sedimentation but may be used in lieu of primary sedimentation where subsequent treatment units are designed on the basis of anticipated screen performance. Selection of screen capacity should consider flow restriction due to retained solids, frequency of cleaning, and extent of cleaning. Where fine screens are used, additional provision for removal of floatable oils and greases should be considered.

(b) Design

A minimum of two fine screens should be provided, each unit being capable of independent operation. Capacity should be provided to treat peak design flow with one unit out of service.

Fine screens should be preceded by a coarse bar screening device. Fine screens should be protected from freezing and located to facilitate maintenance.

2. Grit Removal Facilities

Grit removal is usually accomplished by grit channels or aerated grit chambers. Vortex-type (paddle or jet induced vortex) is another not so common type of device used for grit removal.

(i) Grit Channels

(a) Where Required

In advance of pumping or treating units behind screening devices.

(b) Design and Installation

- *Number of Channels*

  At least two (with one out-of-service, there should be enough capacity in the remaining unit to handle the peak design flow). Provision should be made for isolating and dewatering each unit.

- *Velocity*

  Channels should be designed to control velocities during normal variations in flow as close as possible to 0.3 m/s.

- *Channel Length and Width*

  The length should be adequate to settle 0.2 mm particle with a specific gravity of 2.65 plus 50% allowance for inlet and outlet turbulence. Channel width should be greater than 375 mm.

- *Grit Storage*

  With permanently positioned weirs, the weir crest should be kept 150 to 300 mm above the grit channel invert to provide for storage of settled grit (adjustable weir plates are recommended as they can be moved to prevent the sedimentation of organic solids following grit cleaning).

(ii) Aerated Grit Chambers

(a) Design and Installation

Aerated grit chambers for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, may be designed in accordance with the following parameters:

- *Detention Time*

  2 to 5 minutes at peak design flow rate (the longer retention times provide additional benefit in the form of pre-aeration).

- *Air Supply*

  4.5 to 12 L/m.s, via wide band diffusion header positioned lengthwise along one wall of tank.

- *Tank Dimensions*

  Lower limit of above aeration rates generally suitable for chambers up to 3.7 m deep and 4.3 m wide; wider, or deeper chambers require aeration rates in the upper end of the above range; long, narrow aerated grit chambers are generally more efficient than short chambers and produce cleaner grit; length/width ratio normally is 1.5:1 to 2:1, but up to 5:1 may be used; depth/width ratio 1:1.5 to 1:2.

- *Desired Velocities*

  Surface velocity should be 0.45 to 0.6 m/s.

- *Grit Handling*

  Grit chambers should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage should be provided for grit handling areas. Grit transporting facilities should be provided with protection against freezing and loss of material.

- *Grit Washing*

  Depending upon the method of removal and ultimate disposal, the grit may have to be washed after removal by devices of the type discussed in the previous section.

- *Multiple Units*

  Generally not required unless economically justifiable, or where grit removal method requires bypassing of chamber.

7.3.1.5 Primary Treatment

Primary treatment consists of pre-aeration settling (sedimentation) to remove readily settleable solids, floating materials and scum from raw sewage. This is an important process in sewage treatment, as it reduces the suspended solids content and the load on the biological treatment units.

Sedimentation may be accomplished in horizontal or vertical flow tanks. In a horizontal flow tank, the sewage enters at one end and leaves at the other end. In a vertical flow tank, sewage enters at the centre and flows to periphery of the tank. Sludge settles to the tank floor and is removed mechanically into hoppers where subsequent withdrawal occurs.

Accumulation of scum is to be expected in the primary tank; and a scum baffle/skimmer bar is necessary to prevent the scum from discharging with the effluent.

The tank sizing should reflect the degree of solids removal needed and the need to avoid septic conditions during low flow periods. Sizing of the clarifier should be based on both the average design and peak design flow conditions, and the larger area determined should be used.

Maintenance provisions, including access to equipment, lighting, hose, bibs, etc. should also be provided.

- *Design Criteria*

(i) Minimum water depth - 2.1 m.
(ii) Depth to length ratios (rectangular tank) - 1/10 to 1/30.
(iii) Surface loading for tanks not receiving return sludge; based on average design flow - < 0.47 L/s/m^2.
(iv) Surface loading for tanks not receiving return sludge; based on peak design flow - 0.71 to 1.42 L/s/m^2.
(v) Surface loading for tanks receiving return sludge; based on peak design flow - < 0.47 L/s/m^2.
(vi) Weir overflow rate - 1.74 to 5.21 L/m.s.
(vii) For scum removal, a scum baffle extending at least 150 mm below the surface is necessary close to the overflow weir.

7.3.1.6 Secondary Treatment

1. General

The objective of secondary treatment (mechanical) is to achieve the effluent standards as specified in section 3.1.2, Table 3.1. It may be accomplished in a suspended growth system, a fixed film system or a coupled system. The following sections will discuss the use of two suspended growth systems: continuous-flow activated sludge process, sequencing batch reactors and one fixed film process, rotating biological contactors.

An integral component of the secondary treatment is the secondary clarifier, which will be discussed in Section 7.3.1.8.

2. Suspended Growth Systems

(i) Continuous-flow activated sludge process

(a) General

The activated sludge process and its various modifications may be used where wastewater is amenable to biological treatment. This process requires close attention and competent operating supervision, including routine laboratory control. These requirements should be considered when proposing this type of treatment.

(b) Pretreatment

The minimum level of pretreatment should include grit removal and screening. Primary settling tanks are required unless demonstrated otherwise. If primary tanks are not provided, then the downstream units should be adequately sized.

(c) Types

A number of modifications of the activated sludge process have been developed. The major types of continuous-flow activated sludge processes include: plug flow (conventional), complete mix, extended aeration, contact stabilization and step feed systems.

(d) Sludge Bulking Control

The control of sludge bulking condition should be a main design objective of an activated sludge system. The use of the following process configuration should be considered:

- Plug-flow reactor

- If a complete-mix system is proposed, a selector basin should be constructed upstream of the complete mix basin. Complete-mix activated sludge systems are generally more vulnerable to sludge bulking for treating domestic wastes.

In addition, provision for addition of chlorine or other chemicals to selectively eliminate filamentous bacteria should be included. The chlorine dosage should range from 3 to 15 kg Cl_2 per 1000 kg MLSS/d. The chlorine addition locations should be chosen such that the chlorine is added efficiently and the biomass receives adequate chlorine exposure.

(e) Winter Protection

Due design considerations should be given to minimizing heat loss and to protecting against freezing during winter. Possible design approaches to reduce heat loss during winter conditions include use of diffused aeration instead of surface aeration, reduced surface area by using a deeper tank, tank insulation and provision of covers. If severe climatic conditions are expected, heat loss calculations should be included in the design to assess the aeration basin temperatures under low ambient and wastewater temperatures and low organic loading conditions.

(f) Aeration Basins

- *Sizing*

  The size of the aeration basins should be determined based on the design sludge age using the maximum monthly average BOD loading in the design year. Table 7.2 shows the generally accepted range of sludge age and other parameters such as F/M ratio, mixed liquor suspended solids, aeration tank detention time, aerator loading and sludge recycle ratio for design of the various modifications of the activated sludge process. Consideration should be given to the low loading conditions in the initial operating period.

- *Number of Units*

  Multiple tanks capable of independent operation should be provided for all plants.

- *Dimensions*

  The dimensions of each independent mixed-aeration tank or return sludge reaeration tank should be chosen so as to maintain effective mixing and utilization of air when diffused air is used. Liquid depths should not be less than 3 m, except in special design cases.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid levels. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting,, dissolved oxygen, and air to each aeration tank.

TABLE 7.2
TYPICAL ACTIVATED SLUDGE DESIGN PARAMETERS

Process Modification	Flow Regime	Food to Micro-organism Ratio (g BOD/d) / (g MLSS)[d]	Sludge Age (days)	Mixed-Liquor Suspended Solids (mg/l)	Detention Time (hr)	Organic Loading (g BOD/d m^3 tank volume)	Activated Sludge Return Ratio[c]
Conventional	Plug	0.2 to 0.4	5 to 15	1500 to 3000	4 to 8	350 to 650	0.25 to 0.5
Complete mix	Complete mix	0.2 to 0.6	5 to 15	2000 to 5000	3 to 5	350 to 1900	0.25 to 1.0
Step aeration	Plug	0.2 to 0.6	5 to 15	2000 to 3500	3 to 5	350 to 1000	0.25 to 0.75
Contact stabilization	Plug or complete mix	0.2 to 0.6	5 to 15	1000 to 4000[a] 4000 to 10 000[b]	0.5 to 1.5[a] 3 to 6[b]	500 to 1200	0.25 to 1.0
Extended aeration	Plug or complete mix	0.05 to 0.15	10 to 30	2000 to 6000	10 to 24	150 to 400	0.75 to 1.0
High rate	Complete mix	0.4 to 1.5	5 to 10	6000 to 8000	1 to 3	1600 to 4000	0.25 to 0.5
High-purity oxygen systems	Complete mix reactors in series	0.2 to 1.2	3 to 10	3000 to 6000	1 to 5	1600 to 4000	0.25 to 0.5

Notes:
a Contact tank
b Stabilization tank
c 1.5 to 2.0 is required for nitrifying facilities
d < 0.2 is required for nitrifying facilities

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting,, dissolved oxygen, and air to each aeration tank.

- *Freeboard*

  Aeration tanks should have a freebroad of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each aeration tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the activated sludge process. Provision should be made to isolate each aeration tank without disrupting flow to other aeration tanks.

(g) Activated Sludge Return Equipment

The minimum return sludge rate of withdrawal from the secondary clarifier is a function of the concentration of suspended solids in the aeration tank, the settleability of these solids, and the length of time these solids are retained in the secondary clarifier. The rate of sludge return expressed as a ratio of the average design flow should be variable within the limits set forth in Table 7.2. Separate sludge return lines and pumps should be provided for each clarifier; the system should also be equipped with mechanical or electrical variable speed drive to vary the output of the pump. The pump should be designed for 50 to 125% return of sludge.

Provision should be made in the return lines for the addition of chlorine to the return sludge for controlling sludge bulking.

(h) Waste Activated Sludge Equipment

In designing waste activated control facilities, flexibility should be provided so that the excess activated sludge may be wasted from the return activated sludge lines or directly from the aeration tank. While wasting from the return lines gives a more concentrated sludge, wasting directly from the mixed liquor provides a simpler process control.

The waste activated sludge pumps and pipelines should be sized based on the expected maximum sludge production rates and minimum sludge concentrations. For installations where sludge wastage is not continuous, the sizes of the pumps and pipelines should be increased to handle the sludge wastage during the expected wasting period.

(i) Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- Safety vests
- Lifelines and rings
- Safety poles

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.

Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

(ii) Sequencing Batch Reactors

(a) General

The sequencing batch reactor (SBR) is a fill and draw activated sludge treatment system. It includes a generic system of variable volume activated sludge in which aeration, sedimentation and decant are combined in a single reactor. Consequently, there are no dedicated secondary clarifier or associated return sludge facilities. The SBR technology is suitable for small installations.

(b) Pretreatment

The minimum level of pretreatment should include grit removal and screening.

(c) Types

The SBR systems can be classified under two main types: (1) intermittent feed and intermittent discharge (IFID) and (2) continuous feed and intermittent discharge (CFID).

The IFID systems are sometimes referred to the conventional SBR systems. The common characteristics of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle. The IFID reactor treats the influent wastewater through a succession of operating steps, namely fill, react, settle, draw and idle. The liquid volume inside the SBR increases from a set minimum volume to a predetermined maximum volume during the fill period. Mixing and/or aeration may be provided during this fill step. During the react period, flow to the tank is discontinued and aeration and/or mixing are provided, while sufficient time is allowed for the microbial reactions to take place. During the settle period, quiescent conditions are initiated and the biomass is allowed to flocculate and settle prior to removal. During the draw or decant period, the treated and clarified supernatant is removed from the reactor to the minimum volume level. During the idle period, which is normal component in multireactor installations, biomass is retained in the reactor but no waste is treated. During this period, excess biomass may be removed from the tank to maintain the desired sludge age.

The CFID reactors receive wastewater during all phases of the treatment cycle. Because it has continuous fill, it has no separate fill and idle periods. The CFID reactors always have a prereaction compartment at the influent end terminating in a baffle.

(d) Winter Protection

The winter protection requirements for the SBR systems are higher than the continuous flow activated sludge systems because of the longer total retention times. In addition to the provisions stated for Suspended Growth Systems (subsection 2.i.e), further considerations should be given to the possibility of freezing of equipment and impact of frozen scum on the proposed decanter system.

(e) SBR Basins

- *Sizing*

  The size of the SBR basins should be determined based on the design aerobic sludge age or aerobic food to microorganism (F/M) ratio, using the maximum monthly average BOD loading in the design year. The aerobic sludge age (or F/M ratio) is determined based on the total system sludge age (or F/M) adjusted based on the aerate (aerate fill plus react) in the operating cycle. The aerobic sludge age and F/M ratio should fall within the acceptable range stated in 7.2. The MLSS levels of an SBR change throughout the operating cycle. The selected MLSS levels in calculating sludge age or F/M ratios should correspond to the levels during the react period.

- *Dimensions*

  A key design consideration with CFID systems is minimization of short-circuiting between influent and effluent. The reactor should be rectangular in shape with length to width ratios of at least 2:1. Baffling should also be provided. The length to width ratio is generally less critical for IFID system but the exact dimensions may be affected by the choice of influent distribution system.

- *Liquid Depths*

  The top liquid depths should not be less than 3 m, except in special design cases. In most practical cases, the top liquid depths should range between 4 to 6 m. The bottom liquid depth should be designed based on the required fill volume to handle peak flow conditions. The bottom liquid depths should be decided based on the expected sludge settleability.

- *Number of Units*

  A IFID system must comprise a minimum of two SBR tanks or a storage tank and an SBR tank to accommodate continuous inflow. One CFID reactor is adequate to handle continuous flow. However, multiple tanks capable of independent operation should be provided for all plants.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, sludge wasting, dissolved oxygen, and air to each SBR tank.

- *Freeboard*

  SBR tanks should have a freebroad of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems with provision for chlorine addition or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each SBR tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the SBR process. Provision should be made to isolate each SBR tank without disrupting flow to other aeration tanks.

- *Overflow*

  An overflow system should be provided in the SBR basins to handle extreme flow conditions or equipment malfunction conditions.

(f) Decanter

There are various decanter designs of varying sophistication and complexity proposed by the SBR equipment suppliers. In exposed installations where severe climatic conditions, winter protection should be a major consideration in selecting the decanter design.

Among the recent types are (i) floating decanter (ii) fixed decanter and (iii) mechanically actuated surface skimmer. If a fixed decanter is proposed, longer duration of the settle period should be allowed to ensure that the sludge blanket is located low enough to start each decant cycle.

The decanter should have positive control to prevent solids entry into the decanter during aerate and settle period. The common solids excluding decanters include those (i) incorporating a spring loaded solids excluding valve (ii) physically removed from the mixed liquor except during decant period and (iii) mechanically closed when not in use by a hydraulic or electric motor. If positive control is not provided, the effluent from the decanter should be recycled for at least the first several minutes before discharge. The decanter should have positive control against the entry of scum during the settle period; the decanter should also have a scum baffle to prevent scum from exiting with the effluent.

The size of the decanter should be determined based on the fill volume and the decant period. The decant period is generally **one** half to one hour.

(g) Waste Activated Sludge Equipment

The wasting of excess activated sludge is generally discontinuous for SBR system. The size of the waste activated sludge equipment should be decided based on the expected wastage period. Consideration should be given to the practical number of operating cycles during the working hours of the operator.

(h) Aeration Equipment

When choosing the aeration equipment for a SBR system, consideration should be given to the intermittent aeration conditions and the possibility of diffuser clogging. Because aeration takes place only during part of the operating cycle, the aeration equipment should be sized such that the required oxygen transfer can be provided during the react/fill and react periods. The temporal variation of oxygen requirements should also be considered.

(i) Downstream Facilities

The design of downstream facilities should allow for the intermittent discharge of SBR effluent. Note that the average decant rate is generally higher than the design peak flowrate because of the intermittent discharge. If the decanting devices are of varying flowrate design, the peak flowrate in the beginning of the decant cycle should be used to determine the hydraulic capacity of the downstream facilities. The impact of the intermittent effluent discharge on the downstream facilities such as UV disinfection should be taken into consideration. An effluent equalization basin should be provided, when appropriate.

(j) Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- Safety vests
- Lifelines and rings
- Safety poles

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.
Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

3. Fixed Film Systems

(i) Rotating Biological Contactor

(a) General

The rotating biological contactor (RBC) process may be used where wastewater is amenable to biological treatment.

(b) Pretreatment

Primary clarifiers should be provided ahead of the RBC process to minimize solids settling in the RBC tanks. If the influent contains appreciable amount of sulphide greater than 0.5 mg/L), preaeration should be provided upstream of the RBC process.

(c) Media Types

The media used for RBCs are manufactured of high density polyethylene and are provided in different configurations or corrugated patterns.

The types of media are classified based on the area of media on the shaft and are commonly termed as standard density, medium-density and high-density. Standard density media have surface area of 9300 m^2 per 8.23 m shaft and should normally be used in the lead stages of an RBC process train. Medium and high density media have surface areas of 11 150 to 16 700 m^2 per shaft and should be used only after the second shaft.

(d) Staging

RBC plants should be designed in multiple stages with sufficient operational flexibility to split incoming flows between stages during peak loading periods so as not to exceed loading limitations to the first stage. A minimum of two stages is required.

(e) Design Loading

The typical design loadings for non-nitrifying rotating biological contactors are shown in Table 7.3.

TABLE 7.3
TYPICAL RBC DESIGN LOADING

Organic loading kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	 4 to 10 10 to 17
Maximum organic loading in the first stage kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	 20 to 30 40 to 60
Hydraulic Loading, $m^3/d/m^2$ of disc surface	0.02 to 0.08

(f) Enclosures

Enclosures should be provided for the RBC media to prevent algal growth on the media and minimize the effect of cold weather. Enclosures may be either fabricated individual enclosures or building enclosing several shafts. A building, enclosing the units, is preferable to individual enclosures, due to problems of any repair of the individual enclosures in the winter.

Individual enclosures, if proposed, should be made of material resistant to damage from humidity and corrosion. The exterior of enclosures should be resistant to deterioration from direct sunlight. Access points should be provided at each end of the enclosure to permit inspection of shafts and to perform operation and maintenance.

Enclosures should be removable to allow removal of the shaft assemblies. Access around enclosures should be sufficient to permit suitable lifting equipment access to lift covers and shafts.

Buildings should be designed with provision to remove shafts without damage to the structure. Buildings should also be designed with adequate ventilation and humidity control to ensure adequate oxygen is available for the RBC shafts, provide a safe environment for operating staff to perform normal operation and maintenance and minimize the damage to the structure and equipment from excess moisture. Building material and components should be resistant to corrosion.

(g) Hydraulics

The RBC design should incorporate sufficient hydraulic controls, such as weirs, to ensure that the flow is distributed evenly to parallel units. RBC tank design should provide a means for distributing the influent flow evenly across each RBC shaft. Intermediate baffles placed between treatment stages in the RBC system should be designed to minimize solids deposition. The RBC units should be designed with flexibility for series and parallel operation.

(h) Dewatering

The design should provide for dewatering of RBC tanks.

(i) Shaft Drives

The electric motor and gear reducer should be located to prevent contact with the wastewater at peak flow rates. Variable speed drives should be provided.

(j) Recycle

Effluent recycle after clarification should be provided for small installations where minimum diurnal flows may be very small. Recycle should be considered in any size plant where minimum flows are less than 30 percent of the average daily design flow.

(k) Load Cells

Load cells should be provided for each shaft. A clean water wet load should be derived at startup to provide for biomass measurement after growth occurs.

4. Aeration

(i) Oxygen Requirements

Secondary biological systems should generally be designed to supply oxygen to satisfy the carbonaceons biochemical oxygen demand. However, depending on the local conditions, the designer should also take into account the nitrogenous biochemical oxygen demand and inorganic chemical oxygen demand in calculating the oxygen requirements.

It is likely that nitrification (oxidation of ammonia nitrogen to nitrate nitrogen) will occur during the summer when temperatures are higher; without adequate oxygen, the onset of nitrification can lead to septic conditions and process upsets. Though nitrification is controlled during the summer period by reducing the inventory of MLSS in the aeration tank, some nitrogenous oxygen demand is inevitable and the designer is well advised to provide an allowance for oxygen transfer capability to satisfy these periods of partial nitrification.

Based on the wastewater characteristics, inorganic chemical oxygen demand should also be considered in calculating the oxygen requirements. For instance, hydrogen sulphide will have a demand on oxygen under septic conditions. The designer should evaluate the effect of wastewater septicity and include an allowance for oxygen requirements associated with hydrogen sulphide in the influent wastewater.

a) Carbonaceous Biochemical Oxygen Demand

The carbonaceous oxygen demand may be estimated based on the process oxygen balance using the following formula.

$$R_c = Q\,(S_o - S)(1 + b\,O_c - BY)/(1 + bO_c)$$

where:

R_c	=	mass oxygen required per unit time to satisfy the carbonaceous biochemical oxygen demand.
Q	=	flow rate
S_o, S	=	total carbonaceous oxygen demand of the influent and effluent. The effluent CBOD should be assumed to be zero in the sizing of aeration equipment.
b	=	endogenous decay coefficient
Y	=	true cell yield
O_c	=	solids retention time
B	=	oxygen equivalent of cell mass

As an alternative, the carbonaceous oxygen requirements based on different sludge age are suggested below:

SRT	Oxygen Required (kg O_2 / kg CBOD)
5	1.0
10	1.1
15	1.2
20+	1.3

(b) Nitrogenous Biochemical Oxygen Demand

The nitrogenous oxygen demand may be estimated by the following equation:

$R_n = 4.6 \times Q \times (N_o - N)$

where N_o and N are influent and effluent oxidizable nitrogen respectively. The effluent nitrogen level should be assumed to be zero in the sizing of the aeration equipment.

(c) Inorganic Chemical Oxygen Demand

This oxygen demand is most often estimated based on a stoichiometric calculation.

(d) Spatial and temporal variations

In addition to the total oxygen demand caused by the above sources, the spatial and temporal variations in the demands within the reactor should also be considered in sizing the aeration equipment.

(e) Mixing requirement

In addition to oxygen transfer, sufficient mixing should be maintained such that the biological solids are kept in close contact with the wastewater as shown.

Type of Equipment	Minimum Mixing Energy
Fine bubble diffused aeration	40 $m^3/min/10^3.m^3$ floor coverage
Coarse bubble diffused aeration	20 $m^3/min/10^3.m^2$ floor coverage
Mechanical	20 $kW/10^3.m^3$

(ii) Aeration System Alternatives

The aeration equipment can be classified under two categories. The first category is the diffused aeration system. This system supplies oxygen by introducing air into the wastewater with submerged diffusers or other aeration devices. The equipment under this category can be further divided into three groups: porous diffuser system, nonporous diffuser system and other aeration devices such as aspirators or jet aerators.

The second category is the mechanical aeration system, which supplies oxygen by agitating the wastewater mechanically so as to promote solution of air from the atmosphere. Mechanical aerators are usually divided into two major groups: aerators with a vertical axis and aerators with a horizontal axis. Both groups are further subdivided into surface and submerged aerators.

The selection of the aeration equipment should be based on both cost and nonmonetary considerations. Under cold climatic conditions, special care should be given to freeze protection if surface aerators are proposed.

(a) Diffused Air Systems

Air volume requirements for channel, pumps, or other air-use demands should be added to the oxygen requirements of the activated sludge process.

The specified capacity of blowers, particularly centrifugal blowers, should take into account that the air intake temperature might reach extremes and that pressure might be less than normal. Motor horsepower should be sufficient to handle the minimum and maximum ambient temperature. Piping head loss must also be accounted for.

The blowers should be provided in multiple units, arranged and in capacities to meet the maximum air demand with the largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the plant.

The diffusers should be spaced in accordance with the oxygen and mixing requirements in the basin and should be designed to facilitate adjustments of their spacing without major revision to air header piping. The arrangement of the diffusers should also permit their removal for inspection, maintenance, and replacement without dewatering the tank and without shutting off the air supply to other diffusers in the tank. Slip fittings should not be used. Pipe vibrations should be dampened.

Individual units of diffusers should be equipped with control valves, preferably with indicator markings for throttling and complete shutoff. Diffusers in each assembly should have substantially uniform pressure loss.

Flowmeters and throttling valves should be placed in each header. Air filters should be provided to prevent clogging of the diffuser system.

For further details, the designer should refer to the USEPA publication entitled <u>Fine Pore Diffused Aeration Manual</u>.

(b) Mechanical Aeration Systems

The mechanism and drive unit should be designed for the expected conditions in the aeration tank in terms of the proven performance of the equipment.

Due to high heat loss, consideration should be given to protecting subsequent treatment units from freezing where it is deemed necessary. Multiple mechanical aeration unit installations should be designed to meet the maximum oxygen demand with the largest unit out of service. The design should normally also provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

A spare aeration mechanism should be furnished for single-unit installations.

(iii) Flexibility and Energy Conservation

The design of aeration systems should provide adequate flexibility to vary the oxygen transfer capability and power consumption in relation to oxygen demands. Particular attention should be given to initial operation when oxygen demands may be significantly less than the design oxygen demand. The design should always maintain the minimum mixing levels; mixing may control power requirements at low oxygen demands.

Dissolved oxygen probes and recording should be considered for all activated sludge designs. For larger plants, consideration should be given to automatic control of aeration system oxygen transfer, based on aeration basin dissolved oxygen concentration.

Watt-hour meters should be provided for all aeration system drives to record power usage.

Energy conservation measures should be considered in design of aeration systems. For diffused aeration systems, the following should be considered:

- Use of smaller compressors and more units
- Variable-speed drives on positive-displacement compressors
- Intake throttling on centrifugal compressors
- Use of timers (minimum mixing should be maintained)
- Use of high-efficiency diffusers

For mechanical aeration systems, the following should be considered:

- Use of smaller aerators
- Variable aeration tank weirs
- Multiple-speed motors
- Use of timers

7.3.1.7 Tertiary Treatment

1. General

The objective of the tertiary treatment (mechanical) is to achieve the effluent standards as specified in section 3.1.2, Table 3.2. The tertiary treatment entails nutrient (phosphorus and ammonia) control and effluent disinfection in addition to the reduction of carbonaceous biochemical oxygen demand and total suspended solids. The selection of the most appropriate process is dependent on the following factors:

- Existing process, if any
- Wastewater characteristics
- Space and hydraulic constraints
- Operator's preference

2. Phosphorus control

Phosphorus control can be achieved biologically or chemically. The selection of the most appropriate control method should be made based monetary and process considerations. The important factors which impact on the costs comparison include (1) influent phosphorus levels and loadings (2) chemical costs (3) sludge disposal costs (4) amenability of Biological Nutrient Removal (BNR) retrofit of the existing process. In addition to the monetary comparison, there are process advantages associated with the BNR process which should be considered:

- Improved sludge settleability

- Recovery of alkalinity (for denitrification)

a) Biological Phosphorus Removal (BPR)

BPR is accomplished by a group of organisms that have the ability to uptake quantities of phosphorus in excess of their synthesis requirements when stressed by environmental conditions. This is termed "luxury uptake" and occurs when anaerobic conditions are present in the influent region of a plug flow reactor and aerobic conditions elsewhere. Under these conditions, the phosphorus content of the mixed liquor increases from approximately 1.5 percent to as high as 7 percent solids. Phosphorus is removed via. wasting sludge. The luxury uptake phenomena is actually part of an energy storage cycle that allows the phosphorus accumulating organisms to become active in the anaerobic zone.

There are several factors that can affect the phosphorus removal efficiency of the BPR systems. These factors relate to wastewater characteristics, system design and operational methods. These factors can be divided into the following: (i) environmental factors, such as temperature, D.O. and pH, (ii) Substrate availability as affected by influent wastewater characteristics, the level of volatile fatty acid (VFA) production, and the presence of nitrates, (iii) design parameters, such as system Sludge Retention Time (SRT), anaerobic zone detention time, and aerobic zone detention time (iv) effluent total suspended solids concentration.

There are various process configurations to achieve BPR. The process configurations include A/O, A2/O, Modified Bardenpho, UCT, VIP, Step Bio-P, PhoStrip and Bio-denipho and Trio-denipho processes. Some processes are proprietary. Most processes require different degrees of plant capacity derating to achieve BPR, when retrofitting existing wastewater treatment plants.

It is not the intention of this section to provide step-by-step guidelines for the design of a BNR system. However, adequate design consideration should be given to the following:

- Account for the effect of sidestream returns
- Avoid trapping of Nocardia/foam in the bioreactors
- Provide flexibility of zonal hydraulic retention times (HRTs)
- Provide good D.O. control of all aerobic cells
- Include by-pass capability to reduce unnecessary tankage during initial or low load conditions
- Provide flexibility of recycle sources and destinations
- Pay careful attention to mixing energy and mixer placement
- Avoid secondary release of phosphorus (release without energy uptake)

For additional details, the designer should refer to the USEPA publication entitled Design Manual on Nitrogen Control and Phosphorus Removal.

(b) Chemical Phosphorus removal

Phosphorus in wastewater can be removed chemically by the addition of metallic salts or lime. Lime addition is seldom practised today due to the high chemical usage, problems associated with handling lime and the large volume of sludge generated from lime addition.

The metallic salts commonly used for phosphorus removal are aluminum or iron salts. Iron salts are less common for use in Alberta because of their costs, and limited availability. The common aluminum salts for phosphorus removal include alum, sodium aluminate, polyaluminum chloride and Polyaluminum Silicate Sulphate. Generally, the best removal result is achieved when an organic polymer is added with the metallic salts to assist the precipitation process. Jar tests should be carried out to determine the most appropriate chemicals or their combination and the optimal dosage.

The metallic salts and polymer can be added before the primary clarifier, before or inside the aeration basin and before the final clarifiers. It is a good practice to provide multiple feed points to improve process flexibility. The locations of the feed points should be chosen such that the flow conditions are turbulent to promote good mixing of chemicals.

3. Ammonia Removal

Ammonia can be removed either biologically or chemically using air stripping or breakpoint chlorination. Chemical method for ammonia removal is generally less cost effective and should be considered only under special cases.

The biological removal of ammonia is achieved by the biochemical oxidation of ammonia to nitrate with nitrite as an intermediate. Two autotrophic microorganisms, Nitrosomonas and Nitrobacter are responsible for these reactions. These bacteria grow slower than the BOD removal bacteria and are sensitive to low temperature and adverse environmental conditions. The most important requirement for achieving consistent nitrification is to maintain adequate sludge retention times to prevent these slow growing bacteria from washing out from the biological systems.

Alkalinity is consumed in the nitrification process (7.4 mg of alkalinity as $CaCO_3$ for every mg of NH_4-N nitrified). This loss of alkalinity may have an impact on the design of nitrifying plants treating poorly buffered or low alkalinity wastewater, because the nitrifiers also tend to be pH dependent.

Nitrification can be achieved by either a suspended growth or fixed film process. By incorporating dentrification into the design, approximately two thirds of the oxygen required for nitrification can be recovered.

4. Effluent Disinfection

(i) General

(a) Methods of Disinfection

Bactericides, Viricides and Potential Disinfectants

UV radiation, chlorine and bromine chloride may be used in full scale plants to reduce microorganisims (bacteria and viruses) in the watewater effluent.

Disinfectants

Ozone and chlorine dioxide may be used in full scale plants to reduce microorganisims (cysts, bacteria, and viruses) in the wastewater effluent.

(b) Selection Method

The selection of a disinfection method should be based on both monetary and nonmonetary considerations. In conducting cost comparison, both capital, O&M and total life-cycle costs should be estimated. The nonmonetary considerations should include disinfection effectiveness, state of development, effluent quality impacts, chemical hazards and safety concerns, process complexity and ease of operation and maintenance.

(ii) UV Radiation

(a) General

UV radiation may be used to achieve the disinfection requirements to produce effluent that meets Alberta surface water bateriological quality for recreational waters. At the UV dose commonly applied, this process is effective in inactivating indicator bacteria such as those members of the coliform group, and to a lesser extent the viruses. At this dose, UV is not effective in inactivating protozoa.

(b) Types of UV Systems

UV disinfection systems can be broadly classified under three groups based on the types of lamps used: (A) low-pressure, low-intensity (B) low-pressure, high-intensity and (C) medium-pressure, high-intensity UV systems. The sources of UV radiation of these UV systems are mercury vapour lamps, which are operated at different mercury vapour pressures and discharge currents.

Low-pressure, high-intensity systems are uncommon and not widely used.

A) Low-Pressure, Low-Intensity Systems

Of the disinfection systems using low-pressure, low-intensity mercury lamps, there are three major different reactor designs: open channel, closed chamber and teflon tube. In the open channel systems, the lamps can be oriented either horizontal and parallel-to-flow or vertical. Other reactor configurations such as closed chamber or teflon tube should be considered under special cases only.

B) Medium Pressure, High Intensity Systems

There are three designs using this type of lamps: horizontal and parallel-to-flow, horizontal and perpendicular-to-flow or closed chamber.

(c) Selection of UV Disinfection Alternatives

The factors which affect the selection of the appropriate UV system include:

- Design flowrate
- Wastewater transmittance and suspended solids levels
- Effluent bacteria standards
- Continuous or seasonal disinfection
- Lamp fouling potential
- Land and hydraulic constraints
- Power charges
- Labour costs
- Headloss constraints

Generally, a low-pressure, low-intensity system is suitable for small installations (less than 300 lamps). For major installations, detailed cost comparison of the promising alternatives should be conducted before the UV system is selected.

(d) Influent Characterization

The design of a UV disinfection facility requires a thorough understanding of the characteristics of the influent to the UV facility. The important parameters include flowrates, UV transmittance, total suspended solids and coliform levels. These parameters have direct impact on the UV disinfection efficiency. Other parameters which should also be considered include BOD, ammonia, nitrate, iron, hardness, oil and grease and wastewater temperature. They either have indirect impact on disinfection efficiency or direct impact on lamp fouling.

When a UV disinfection facility is added to an existing treatment plant, the historical data regarding effluent qualities should be studied. Statistical and probabilistic analyses should be conducted to establish the expected worst-case operating conditions as the design basis. Measurement of UV transmittance levels should also be conducted.

If UV disinfection is proposed for a new plant, the effluent qualities of similar installations and available published information may be used as the design basis; or pilot plant studies shoud be considered. The presence and nature of any industrial discharge and the raw water source and quality should be considered. The effluent UV transmittance depends on the type and degree of treatment. Generally, a higher degree of treatment gives a higher light transmittance of the effluent. The UV transmittance from a suspended growth process is generally higher than that from a fixed film process. Typical ranges of UV transmittance values for various secondary processes are shown in Table 7.4.

TABLE 7.4
RANGE OF UNFILTERED UV TRANSMITTANCE

Treatment Processes	Typical Range
Conventional activated sludge	35 to 65 percent
Fixed film process	30 to 50 percent
Lagoon	30 to 50 percent
Secondary plus filtration	50 to 70 percent

(e) Assessment of Minimum UV Dose Requirements

UV dose is defined as the product of the intensity of radiation (microwatts per square centimeter) and the length of time (seconds) during which the wastewater is exposed to UV radiation. The minimum UV dose requirement is dependent on the design influent characteristics and the effluent coliform standards.

The minimum UV dose requirements may be determined using the following methods:

- *Pilot Testing*

  Pilot testing should be conducted if (A) the wastewater treatment plant (WWTP) is treating significant industrial wastes (B) it is designed for major installations or (C) a less common UV system is considered. The pilot testing programme should include an intensive test programme to develop the dose response relationship of the UV system and an routine test program to assess the lamp sleeve fouling potential.

- *Disinfectability Studies*

  The disinfectability of the wastewater may be assessed in a laboratory using a collimated beam test apparatus. The results may be used to develop the dose response relationship under ideal laboratory conditions. Considerations should be given to allow for the less perfect hydraulic design of the full-scale UV reactor.

- *Mathematical Models*

  The minimum dose requirements may be determined using mathematical models such as those developed by USEPA or WERF. When properly calibrated using the data collected from the pilot studies, these models will provide a useful means for estimating the required dose under the design influent conditions. If the models are not calibrated using the actual wastewater characteristics, adequate safety margin should be allowed in the design.

- *Design and Operating Data from Similar Installations*

  The design and historical operating data of some existing UV installations may be used as the design basis. The data to be used in analyses are the influent and effluent coliform data, TSS levels, UV transmittance and flowrates. Lamp cleaning frequency records should also be reviewed. The important criteria for selecting the similar installations include the degree of treatment, the types of treatment processes and industrial components of the wastewater.

(f) Development of Design Dose and Lamp Requirements

The UV dose used to design the UV facility should be adjusted to account for aging and fouling effects of the UV lamps. The minimum UV dose developed based on clean and new lamps should be increased by the following formula:

Design UV dose = minimum UV dose/Fp/Ft

where F_p = The lamp output reduction factor. This factor is the fraction of initial output the lamp is expected to have at the end of its useful life.

F_t = The quartz sleeve transparency reduction factor. This factor is the fraction of lamp emission that is transmitted through the quartz sleeve immediately before the sleeve is cleaned. It includes the UV output loss due to the clean quartz sleeve and the losses due to the deposits on the sleeve (fouling).

Table 7.5 shows the suggested correction factors for the low pressure UV systems. The appropriate correction factors for the high intensity UV systems should be decided from the full- or pilot-scale operating data.

TABLE 7.5
DESIGN CORRECTION FACTORS

UV Systems	F_p	F_t (1)
Low-pressure, low-intensity	0.65	0.71

Note:
The power correction due to loss in transmittance is based on clean quartz sleeve loss of 11 percent and the transmittance losses due to sleeve fouling of 20 percent for low pressure systems.

The lamp requirements of the UV facility should be calculated based on the design UV dose and the design peak flowrate. Standby UV disinfection capacity is generally not required at peak flow conditions to account for units out of service. UV disinfection equipment cleaning or maintenance procedures that require removal of equipment from service should be able to be completed during the expected low flow conditions. Where peak flows are frequent or unpredictable, or the installations only have limited lamps, standby equipment during peak flow conditions should be provided.

(g) Lamp Arrangements

UV lamps are generally grouped in units of modules. The modules are then grouped into lamp banks. Multiple lamp banks are placed in a UV channel and the complete UV facility consists of one or more UV channels.

The choice of UV lamp numbers in a module is generally limited by the design of the UV system. The number of UV lamps in a low-pressure, low-intensity system of horizontal lamp design may range from 2 to 16 lamps. The number of UV lamps in a low-pressure, low-intensity system of vertical lamp design is typically 40. If a 16 horizontal lamp module or a 40 vertical lamp module is proposed, a mechanical lifting device should be installed to facilitate lamp removal and installation.

The numbers of UV modules in a UV bank are generally determined by hydraulic considerations. The appropriate number of UV modules (and number of lamps per module for horizontal lamp system) should be selected so as to ensure that the UV reactor is conducive to plug flow.

The number of UV banks in a channel should be at least 2 or 4 for horizontal and vertical lamp systems, respectively to maintain plug flow characteristics and minimize short-circuiting. For low-pressure, low-intensity systems, two UV channels should be provided to maintain system reliability.

In designing the lamp arrangement, a main important consideration is to facilitate flow pacing. The lamps should be arranged in a such a way that the lamp banks can be turned on and off easily to match the expected disinfection needs. Some newer vertical-lamp systems are designed such that individual rows of lamps within a given module may be turned on and off based on flowrate signals.

(h) Reactor Designs

- *Inlet and Outlet*

  UV disinfection reactors should be designed with inlet channel approach and outlet conditions that promote plug flow within the system. To ensure proper inlet and outlet flow conditions, the following criteria are suggested:

  - Unobstructed approach channel length before first UV bank not less than 2 times channel water depth or 1.2 m.

  - Unobstructed downstream channel length following last UV bank before water level control device not less than 2 times channel water depth or 1.2 m.

  - Spacing between UV banks = minimum spacing required for maintenance and access

- *Flow Distribution*

  If more than one UV channel is provided, a positive flow distribution system such as weirs should be used to ensure equal flow splitting. The influent chamber should be sized such that the headloss along the chamber is less than one tenth of the headloss at each channel under the expected range of flow conditions.

- *Velocity Distribution*

  For low-pressure, low-intensity system, perforated stilling plates should be provided upstream of the first lamp UV lamp banks to ensure uniform velocity distribution.

- *Water Level Control*

  Water depth control devices should be provided in the low pressure, low-intensity systems to maintain a constant water level (plus or minus 2.5 cm) under the expected flow ranges. Weighted adjustable flap gates are generally suitable. If the flowrates during the initial years of operation are significantly lower than the design flowrates, a fixed weir should be installed initially. The water depth control for a medium-pressure, high-intensity system is less critical because its main function is to ensure full submergence of the reactors at all times. Weighted adjustable flap gates and adjustable weir gates are acceptable.

- *Isolation Gates*

  Sluice or weir gates should be provided upstream and downstream of the UV banks to isolate the UV channel when maintenance is needed. Flap gates which may leak under low flow conditions should not be used as isolation gates.

- *Drainage*

  A drainage (mud) valve should be provided in each UV channel for dewatering.

- *Hydraulics*

  Adequate hydraulic head should be allowed for in the design of the UV facility. The main hydraulic head losses occur at the inlet for flow splitting and at the outlet gates for free flow discharge. The losses at the UV banks and reactors are generally small when compared with other losses.

- *Flood Level*

  The top level of the UV process area should be located above the 1:100 year flood level. The channel depths should be checked such that the UV modules can be conveniently lifted up from the main process area.

(i) Lamp Cleaning

Adequate facilities should be provided to facilitate cleaning of lamp sleeves. In addition to the cleaning chemical system, appropriate washdown area and lifting devices should be provided.

For low-pressure, low-intensity systems, lamp cleaning can be performed manually or with a dip tank for small installations. In large installations, a cleaning chemical (acid) bath should be provided so that one lamp bank can be cleaned at a time. If in-channel lamp cleaning system is proposed, suitable concrete lining and isolation gates should be provided. Gates should preferably be of stainless steel or fibreglass.

For medium-pressure, high-intensity systems, cleaning of lamps may be performed by the built-in wiping mechanisms, as included in some designs. If only a mechanical wiper is provided, provision should be made **for** cleaning the lamps chemically on a regular basis.

(j) Screens

Screens may be required immediately upstream of the UV facilities to remove algal clump or other objects which may impact UV disinfection performance, or damage the lamps. Screen openings may range from 2 to 13 mm, depending on the selected types of UV systems, size of installations, the expected degree of algae problem and availability of operating resources. Mechanical screens should be considered for large installations.

(k) Ballasts

The ballasts should be compatible with the proposed lamp type. If possible, the more energy efficient electronic ballast should be specified.

(l) Control and Instrumentation

The choice of the most appropriate control system for a UV facility is dependent on the size of the installation, the available operating staff and the control system for other WWTP facilities. For major installations, the functions of a UV disinfection control system should include the following:

- Activate and deactivate UV banks and channels based on the disinfection needs

- Activate and deactivate lamp cleaning mechanism (for medium-pressure, high-intensity systems)
- Monitor influent characteristics and equipment operation status
- Generate alarms
- Monitor UV intensity

The major parameters to be monitored continuously in the control system may include flow rate, UV transmittance, UV intensity, water levels, UV output and lamp/bank status. Other important information for process control such as influent and effluent coliform levels and total suspended solids levels will be collected by taking samples manually.

For small installations, the minimum function of a control system should include flow pacing of the UV lamp banks to optimize the UV dose. While overdosing of UV radiation may not create any harmful environmental byproducts, it will increase the O&M costs necessarily by wasting power and reducing useful lamp life.

(m) Housing

The need for housing of the UV facility should be decided on a case-by-case basis. Under extreme climatic conditions, housing is generally desirable for low-pressure systems where manual lamp cleaning is required.

If housing is provided, attention should be paid to humidity control and ease of equipment transport into and within the building.

(n) Sequencing Batch Reactor

If the UV disinfection is proposed with a sequencing batch reactor system, consideration should be paid to the intermittent effluent discharge. The design flowrates should be decided based on the decant flowrates. If decant system is not designed to have constant-flow design, the maximum flowrate during the decant cycle should be used to size the UV disinfection. It may be appropriate to provide an upstream equalization basin to provide continuous flow through the UV system.

The number of expected on-off operating cycles of the UV lamp banks should be estimated. If the expected number of on and off is high, provisions should be made of an equalization basin upstream of the UV facilities or recycle pumps downstream of the UV facilities.

(o) Safety

Three safety issues should be addressed fully in the design of a UV disinfection facility. They are (1) exposure to UV radiation and (2) electrical hazards and (3) handling of acids.

Exposure to UV radiation may affect the eyes with a temporary painful condition known as conjunctivitis or "welder's flash". Bare skin will also be burned upon exposure to UV at these wavelengths. For the low-pressure, low-intensity UV systems, eye shields should be provided in the UV channels. The use of chequer plate instead of open floor grating should also be considered. The medium-pressure, high-intensity systems must be completely enclosed. All UV systems must be equipped with safety interlocks that shut off operating modules if they are removed from the channel.

The safety requirements for handling high-voltage electricity should be followed. All the electrical components should be designed for submergence or located above the 1:100 flood levels. Electrical hazards should be minimized by the inclusion of ground fault interruption circuitry with each operating module.

Due considerations should be given to the cleaning chemical tank design such that the chance of accidental falling into the tank is minimized. Handrails should be provided when needed. Safety shower and eye wash facilities should be provided in the chemical bath and chemical handling areas.

(iii) Ozonation

(a) General

Ozonation is generally applied only to effluents that are nitrified, highly clarified (filtered) or both. This method may also be considered for unfiltered secondary effluents when a low-cost source of oxygen is available, e.g., at a pure oxygen activated sludge WWTP.

(b) Source of Ozone

Ozone must be generated on site because it is chemically unstable and decomposes rapidly to oxygen after generation. The most efficient method of producing ozone is by electrical discharge using either air or pure oxygen.

(c) Design Requirements

The design requirements for ozonation systems should be decided based on pilot testing or similar full-scale installations. As a minimum, the following design factors should be considered:

- Ozone dosage
- Dispersion and mixing of ozone in wastewater
- Contactor design
- Control of off-gas

(d) Gas Selection and Preparation

Ozone may be generated from air, oxygen-enriched air or high purity oxygen. The concentration of ozone produced increases 2 to 2.5 times when air is replaced by high-purity oxygen at the same gas flow rate. The selection of feed gas should be decided based on cost and other considerations. Generally, the use of pure oxygen is not economical unless high-purity oxygen is required elsewhere in the treatment facility.

Regardless of whether air or high purity oxygen is used as the feed gas, gas preparation is required to ensure it is free of oil, dust and moisture. When ambient air is used as the feed air, the following processing and control units should be provided prior to ozone generation:

- Filter - 5 µm modular or fabric filter
- Pressurizer - blower, compressor
- After cooler
- Oil coalescer - if an oil-free pressurization device is not used
- Refrigerant drier - to reduce the size of desiccant drier
- Desiccant drier - silica gel, activated alumina, or crystalline zeolite
- Filter - 99 percent efficient at 1.0 µm size
- Hygrometer
- Gas flow meter
- Pressure release valve - in high pressure systems

When oxygen enriched air is used, the gas preparation units are similar to those required for ambient air except that the desiccant drier is not needed and replaced by a pressure swing separator.

In the case of high-purity oxygen, the following units should be provided prior to ozone generation:

- High-purity oxygen source -cryogenic plant, pressure swing absorption unit or oxygen cylinders
- Pressurizer - blower, compressor
- After cooler
- Oil coalescer - if an oil-free pressurization device is not used
- Refrigerant drier - to reduce the size of desiccant drier
- Desiccant drier for recycled oxygen and
- Filter - 99 percent efficient at 1.0 µm size
- Pressure release valve - in high pressure systems

(e) Ozone Generation

- *Types*

  The ozone generators may be broadly classified based on their power supply, i.e., low frequency (60 Hz), medium frequency (up to 600 Hz) and high frequency. These types may be further divided based on the cooling media (air, water or both) and the physical arrangements of the dielectrics.

  Small systems (less than 450 kg/d) generally call for the application of low frequency water cooled units. In larger systems, especially those incorporating high purity oxygen process, the added cost and complexity of higher frequency generators with associated chilled water cooling systems may be justified.

- *Sizing*

  The total capacity of the ozone generators should be decided such that the required applied ozone dosage can be delivered under peak flow condition. The applied ozone dosage should be calculated based on the required absorbed ozone dosage and the minimum ozone transfer efficiency at the expected ozone dosage and wastewater quality conditions.

  Standby ozone generator capacity is generally not required at peak conditions. Ozone equipment maintenance that requires removal of equipment from service should be able to be completed during expected low flow or dose requirement conditions. Where peak flows are frequent or unpredictable, stand-by equipment during peak flow condition should be considered.

  The number and size of generator units should be decided such that the system can satisfy both the maximum and minimum ozone production rates under the expected operation conditions.

(f) Ozone Contacting

The ozone contacting basin should be designed such that there is an efficient mass transfer of ozone out of gas bubbles into the bulk liquid and sufficient time for disinfection. Common contactor types include:

- Diffused bubble (concurrent and countercurrent)
- Positive pressure injection
- Negative pressure (venturi)
- Mechanically agitated
- Packed tower

For diffused bubble systems, the contactor should be at least 6 m deep for secondary effluent at an applied ozone dosage of less than 6 mg/L and an elevation of approximately 1000 m in Alberta. The contactor may be deeper if the wastewater to be disinfected is of higher quality, if the applied ozone dosage is higher or if the plant is located at a higher elevation.

Multiple staged ozone contactors should be provided to minimize the effect of short-circuiting. A minimum of three and preferably more stages should be provided. Each stage should be positively isolated from the other to simulate plug flow characteristics and minimize the potential for short-circuiting. The minimum contact time should be 6 minutes and preferably 10 minutes at design flow rates.

The off-gases from the contact chamber must be treated to destroy any remaining ozone. The product formed by destruction of the remaining ozone is pure oxygen, which may be recycled if pure oxygen is being used to generate the ozone.

(g) Housing

If a building is not provided separately for ozone generation equipment, a gas-tight room should be constructed to separate the ozone equipment from other part of the building. Doors to the ozone generation room should open only from the outside of the building and should be equipped with panic hardware.

At least two means of exit should be provided from each separate room. All exit doors should open outward.

A clear glass, gas tight window should be installed in an exterior door or interior wall of the ozone generation room to permit the ozone generator to be viewed without entering the room.

(h) Ventilation

For ozonation system rooms, continuous mechanical ventilation should be provided to maintain at least six air changes per hour. The entrance to the air exhaust duct from the room should be near the floor and the point of discharge shall be selected such that it will not contaminate the air inlet of any buildings or inhabited areas.

(i) Corrosion Protection

The selection of material should be made with due consideration for ozone's corrosive nature. Only materials at least as corrosion-resistant to ozone as grade 304 L or 316 L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasterized PVC may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.

Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for ozone service, with a minimum number of joints. Piping should be well supported and protected against temperature extremes.

(j) Safety

The safety issues that should be addressed fully in the design of ozonation system include (1) exposure to ozone (2) noise and (3) electrical hazards.

The occupational exposure of ozone should be controlled such that workers will not be exposed to ozone concentrations in excess of 0.2 mg/m^3 for 8 hours or more per workday, and that no worker be exposed to a ceiling concentration of ozone in excess of 0.6 mg/m^3 for more than 10 minutes.

All ozone systems should be provided with an ambient ozone monitor or monitors which are set up to measure the ozone concentration at potential ozone-contaminated locations within the plant. The monitors should be set up to sound an alarm when the ozone concentration reaches 0.2 mg/m^3 and should be set up to shut down the ozone system when the concentrations exceed 0.6 mg/m^3.

Eye-wash basins should be provided to enable the operator to rinse ozone from the eyes, if needed. The basins should be located in the outside of the ozone generator room.

Some ozone generation systems are classified as noisy installations. Generally, the main source of noise is the feed-gas compressor. Wherever practical, the feed-gas compressors should be isolated in a sound insulated room.

The ozone generators generally require high power consumption. All safety requirements regarding handling of high voltage power should be followed.

(iv) Chlorine

(a) General

Chlorine is an effective disinfectant. Potential drawback to its use include:

- Toxicity to aquatic, estuarine and marine organisms
- Generation of harmful chlorinated disinfection byproducts
- Safety concerns during transportation, storage and handling**,** particularly with gaseous chlorine

If chlorination is proposed, due consideration should be given to the above factors. A dechlorination system should be provided.

(b) Forms of Chlorine

Chlorine may be added to the wastewater in the form of liquid/gaseous chlorine or sodium hypochlorite.

(c) Design Requirements

In sizing a chlorination system, the following factors should be considered:

- Contact time
- Concentration and type of chlorine residual
- Mixing
- pH
- Suspended solids levels
- Temperature
- Coliform levels
- Ammonia concentrations

The design should provide adequate flexibility in the chlorination and control system to allow controlled chlorination at the expected flow ranges in the design period. Special consideration should be given to the chlorination requirements during the initial years of operation to ensure the chlorination system is operable at less than design flows without over-chlorination.

(d) Chlorine Addition

Chlorine should be added into the wastewater where good mixing is achieved at all times. Mixing may be accomplished mechanically or hydraulically.

When mechanical mixing is proposed, the following criteria apply:

- A mixer-react unit is necessary that provides 0.1 to 0.3 minutes contact.

- Chlorine should be injected just upstream of the mixer with a diffuser

- The minimum mixer speed should be 50 revolutions per minute

- The diffuser should be set at least two feet below the minimum wastewater level at low flows

- Turbulent flow after complete chlorine mixing should be avoided to prevent chlorine stripping.

Hydraulic mixing should be achieved based on the following criteria:

- Pipe Flow

  - A Reynolds number of greater than or equal to 1.9×10^4 should be achieved at all flow rates. Hydraulic jumps or baffles may be used to create turbulence.

  - A diffuser, with orifice velocities of 5 m/s at peak flows should be provided.

  - The diffuser should be set as deep as possible and at least two feet below minimum wastewater level at low flows.

- Open Channel

  - A hydraulic jump with a minimum Froude number of 4.5 is necessary to provide the adequate hydraulic mixing. Multiple points of chlorine injection should be provided because the jump location may change with changes in flowrates. A parshall flume is not a satisfactory location for hydraulic mixing.

(e) Contact Basin

Contact chambers should be sized to provide sufficient retention time for the effluent to meet the required bacteriological quality.

The contact chambers should be baffled to minimize short-circuiting and backmixing. Baffles should be constructed parallel to the longitudinal axis of the chamber with a minimum length-to-width ratio of 40:1. Side water depths should range between 2 to 5 m.

(f) Dechlorination

Dechlorination may be achieved by the use of detention ponds or by the addition of sulphur dioxide or sodium metabisulphite to the chlorinated effluent.

The required sulphur dioxide dosage for dechlorination is 1 mg/L SO_2 for 1 mg/L chlorine residual. Reaction time is essentially instantaneous. Detention time requirements are decided based on the time necessary to ensure complete mixing of the sulphur dioxide. To ensure continuous compliance of the maximum chlorine residual requirements, over-dechlorination followed by reaeration should be considered. Continuous monitoring of the effluent would be a requirement.

(g) Chlorination equipment and chlorine room design requirements

For details of chlorine equipment requirements and chlorine room design requirements, see sections 4.1.5.2 and 4.1.5.3.

7.3.1.8 Secondary Clarifier

Design of a secondary clarifier for suspended growth systems is different from the design of secondary clarifier for fixed growth systems, in that, to perform properly while producing a concentrated return flow, a suspended growth secondary clarifier should be designed to meet thickening as well as solids separation requirements. Since the rate of recirculation of return sludge from the final settling tanks to the aeration is quite high in activated sludge processes, surface overflow rate and weir overflow rate should be adjusted for the various processes to minimize problems with sludge loadings, density currents, inlet hydraulic turbulence, and occasional poor sludge settleability. The size of the settling tank should be based on the larger surface area determined for surface loading rate and solids loading rate.

Design criteria for secondary clarifier is detailed in Table 7.6.

TABLE 7.6
SECONDARY CLARIFIER DESIGN CRITERIA

Treatment Process	Surface Loading Rate at Peak Design Flow[1] (L/s/m^2)	Peak Solids Loading Rate[2] (kg/d/m^2)	Minimum Water Depth (m)	Weir Overflow Rate[3] (L/s/m)
RBC Trickling filters	0.56	-	3.0	2.9 to 4.3
Conventional-activated Sludge; Contact Stabilization	0.56*	245	3.7**	2.9 to 4.3
Extended Aeration	0.47	171	3.7	2.9 to 4.3

1 Based on influent flow only. Lower loading rate should be used for nitrifying plants and those with chemical addition for phosphorus removal

2 Clarifier peak solids loading rate should be computed based on the maximum day design flow plus the maximum return sludge rate requirement and the design mixed liquor suspended solids (MLSS) under aeration.

3 Weir overflow rate would increase with increasing plant capacity.

* Plants needing to meet 20 mg/L suspended solids should reduce surface loading rate to 0.47 L/s/m^2.

** Greater water depths are recommended for clarifiers in excess of 372 m^2 surface area. Less than 3.7 m water depths may be adequate for package plants with average design flow less than 100 m^3/d.

7.3.1.9 Laboratory Requirements

All treatment works should include a laboratory for making the necessary analytical determination and operating control tests, except where satisfactory off-site laboratory provisions are made to meet the operating approval monitoring requirements. The laboratory should have sufficient size, adequate ventilation (particularly where furnace and fume hoods are used for solids and sludge analysis), bench space, equipment, and supplies to perform all self-monitoring analytical work required by the operating approval, and to perform the process control tests necessary for good management of each treatment process included in the design.

The laboratory arrangement should be sufficiently flexible to allow future expansion should more analytical work be needed. Laboratory instrumentation and size should reflect treatment plant size, staffing requirements, and process complexity. Experience and training of plant operators should also be assessed in determining treatment plant laboratory needs.

Before undertaking the detailed design of the laboratory facility, contact should be made with AEP to confirm the testing requirements.

7.3.1.10 Flow Measurements

1. Location

Flow measurement facilities should be provided to measure the following flows:

(i) Plant influent and effluent flow;

(ii) Plant and process unit bypasses; and

(iii) Other flows such as return activated sludge, waste activated sludge, recirculation, and recycle required for plant operational control.

2. Facilities

Indicating, totalizing, and recording flow measurement devices should be provided for all mechanical plants. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

3. Hydraulic Conditions

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

7.3.1.11 Colour Codes

Refer to Table 7.7 for recommended colour coding for wastewater treatment plant piping.

TABLE 7.7
RECOMMENDED COLOUR CODING FOR WASTEWATER TREATMENT PLANT PIPING

Piping to be Identified	Basic Colour	Bands	
		No.	Colour
Raw Wastewater	Brown	-	-
Primary Settled Wastewater Effluent	Brown	1	White
Secondary Settled Wastewater Effluent	Gray	-	-
Sludge Lines			
Raw Wastewater	Black	-	-
Primary Sludge	Black	1	White
Secondary Sludge	Black	2	White
Digested Sludge	Black	3	White
Digested Liquor	Black	1	Brown
Natural Gas	Orange	-	-
Digester Gas	Orange	1	Black
Chlorine Gas	Yellow	-	-
Chlorine and Water	Pink	1	Yellow
Chlorinated Effluent	Grey	1	Yellow
Electrical	Purple	-	-
Compressed Air	White	-	-
Heating	Silver	-	-
Fire Protection	Red	-	-
Potable Water	Blue	-	-
Untreated Water	Dark Green	-	-

Notes

1. Entire length of pipe to be painted in basic colour.
2. Bands, if required, are to be placed as follows:
   (a) at 9 m intervals, and/or
   (b) where the pipe enters and leaves a room.
3. Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

7.3.2 Aerated Lagoons

7.3.2.1 General

The aerated lagoon system is a biological treatment process with long retention time (compared to mechanical systems) and large capacity. The system consists of one or more "complete mix cell" and one or more "aerated cell."

The complete mix cell is designed to provide enough oxygen transfer to satisfy the applied CBOD loading and to maintain a uniform solids concentration. The aerated cell is designed to satisfy the applied CBOD loading while maintaining an adequate uniform dissolved oxygen level in the cell. There is no attempt in the design of the aerated cell to provide complete solids mixing, therefore solids are allowed to settle in the cells to undergo anaerobic decomposition.

7.3.2.2 Design Approach

In general, the following factors should be considered in the design of the aerated lagoons:

- CBOD removal and effluent characteristics;
- Temperature effects;
- Mixing requirements;
- Oxygen requirements; and
- Solids separation

1. CBOD Removal and Effluent Characteristics

CBOD removal and the effluent characteristics are estimated using a complete mix hydraulic model and first order reaction kinetics. The complete mixed model using first order kinetics and operating in a series with 'n' equal volume cells is given by:

$$\frac{L_e}{L_i} = \frac{1}{\left[1 + \frac{K_t T}{n}\right]^n}$$

where:

L_e	=	Effluent BOD, mg/L
L_i	=	Influent BOD, mg/L
K_t	=	Reaction rate coefficient at t°C, day^{-1}
T	=	Total hydraulic retention time in lagoon system, days
n	=	Number of ponds in series

The selection of the reaction rate coefficient is critical in the design of the lagoon system. All other considerations in the design will be influenced by this selection. If possible, a design K_{20} should be determined for the wastewater in pilot or bench scale tests; experiences of others with similar wastewaters and environmental conditions should also be evaluated. Reaction rate coefficient K_{20} may vary from 1.5 day^{-1} for complete mix cell to 0.37 day^{-1} for aerated cell.

When using the complete mix model, the number of cells in series has a pronounced effect on the size of the aerated cell required to achieve a specific degree of treatment. The reactor required to achieve a given efficiency may be greatly reduced by increasing the number of cells in series.

2. Temperature Effects

The influence of temperature on the reaction rate is expressed by the equation:

$$K_t = K_{20}\,\theta^{t-20}$$

where:

K_t = Reaction rate coefficient at t°C, day $^{-1}$
K_{20} = Reaction rate coefficient at 20°C, day $^{-1}$
t = Wastewater temperature, °C
θ = Temperature activity coefficient (varies from 1.04 to 1.1 for aerated lagoons, with typical value of 1.065)

3. Mixing Requirements

Aeration is used to mix the pond contents and to transfer oxygen to the liquid. There is no rational method available to predict the power input necessary to keep the solids suspended. The best approach is to consult equipment manufacturers' charts and tables to determine the power input needed to satisfy mixing requirements. Power of 6-10 w/m^3 of the cell volume is frequently used and these values can be used as a guide to make preliminary estimates of power requirements, but the final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer.

For a complete mix cell, in comparing the power requirements for both, to maintain solids in suspension and to meet the oxygen demand, it would soon become evident that the mixing requirements would control the power input to the system.

After determining the total power requirements for a cell, the diffusers/aeration units should be located in the cell so that there is an overlap of the diameter of influence providing complete mixing.

4. Oxygen Requirements

Oxygen requirements are estimated using equations based upon mass balances, and there are several rational equations available to estimate the oxygen requirements for lagoon systems, however, the use of the CBOD entering the pond as a basis to estimate the biological oxygen requirements, is as effective. Approximately 1.5 kg to 2.0 kg of oxygen is required to remove 1 kg of CBOD in the aerated lagoon system.

As for the other mechanical systems, monitoring of aerated lagoons include adjustment of aeration devices to control dissolved oxygen to be greater than 2 mg/L in the aeration basin during peak loading conditions. Lagoons with odour problems should be corrected by increasing the air supply.

5. Solids Separation

For systems with continuous discharge to a receiving stream, a polishing cell having a minimum hydraulic retention of five days, based on summer average daily design flows, should be provided. Polishing cells are not required for systems having storage facilities with intermittent discharges.

7.3.2.3 System Design (Aeration)

The preceding section outlined the general design approach for designing aerated lagoon systems. This section outlines the procedure for the determining the aeration requirement for mixing (Step 1) as well as for satisfying the applied CBOD loading (steps 2 to 6) in a step format.

Step 1: Determine aeration requirement for mixing in the complete mix cell. As indicated earlier, a power input of 6 - 10 w/m^3 per cell volume may be assumed to make the preliminary estimates, however the final capacity should be based on guaranteed performance by an equipment manufacturer.

Step 2 - Calculate the CBOD reduction in each cell under both summer and winter conditions.

Modifying the formula under section 7.3.2.2(1) for a single cell, CBOD reduction (complete mix cell) as a % of the average daily design CBOD load (E_1);

$$E_1 = \frac{K_t T}{1 + K_t T} \times 100$$

CBOD reduction (aerated cell) as a % of the average daily design CBOD load (E_2);

$$E_2 = \frac{K_t}{1 + K_t T} \times E_1 \times 100$$

CBOD reduction (overall) $E_3 = E_1 + E_2$

K_t values can be determined utilizing the formula noted in section 7.3.2.2(2).

Insert the calculated % reduction values on line 1 in the summary Table 7.8.

Step 3 - Calculate the CBOD removal (Kg) in each cell during both summer and winter conditions as follows:

CBOD removed in complete mix cell = average daily design CBOD load x E_1 (summer and winter).

CBOD removed in Aerated Cell = average daily design CBOD load x E_2 (summer and winter).

Insert the calculated CBOD removed in each cell on line 2 in the summary Table 7.8 and complete line 3.

Step 4 - Calculate the Actual Oxygen Required (AOR) in Kg/hour. This is determined by multiplying the calculated CBOD removal rate as noted on line 3 in the summary table by the oxygen to CBOD ratio (oxygen required in Kg per Kg of CBOD removed). As indicated in the foreceding section, an oxygen to CBOD ratio of 1.5 to 2.0 could be used when designing a system handling typical domestic wastewater.

Insert the AOR on line 4 in the summary Table 7.8.

Step 5 - Calculate the Standard Oxygen Required (SOR) in Kg/hour for each cell and for the total system during summer and winter conditions. This requires determination of the oxygen mass transfer ratio (ϕ) which is utilized to correct the rate of oxygen transfer under standard conditions to the rate of oxygen transfer to the wastewater under site conditions. The formula for determining the oxygen mass transfer ratio is as follows:

$$\phi = \frac{\alpha(\beta C_s - C_L)\gamma^{t-20}}{C_s}$$

where:

ϕ = oxygen mass transfer ratio

α = correction factor used to estimate the actual oxygen transfer in wastewater versus the oxygen transfer in low total dissolved solids water generally used for rating aeration devices. α values of 0.6 to 0.8 are generally used for design.

β = correction factor used to correct the test system oxygen transfer rate for differences in oxygen solubility due to constituents in the wastewater such as salts, particulates and surface active substances. Values range from 0.7 to 0.98. A value of 0.95 is commonly used in the absence of experimental verification.

γ = factor for correcting the oxygen mass transfer rate at temperatures other than 20^{o}C. Typical values are in the range of 1.015 to 1.040. A value of 1.024 is typical for both diffused and mechanical aeration devices.

C_s = solubility of oxygen at site conditions (i.e., site temperatures and barometric pressure) - mg/l.

C_L = minimum required dissolved oxygen concentration in the treated effluent - (2 mg/L during peak loading conditions).

t = wastewater temperature ^{o}C

Step 6 - Calculate the SOR required in Kg/hr for each cell and for the total system under both winter and summer conditions as follows:

SOR for each cell = AOR/ϕ

Total SOR = sum of the SOR for all cells

Insert the SOR values on line 6 in the summary Table 7. 8.

TABLE 7.8
AERATION SYSTEM DESIGN
SUMMARY TABLE

Line No.	Summer			Winter		
	Mix Cell	Aerated Cell(s)	Total	Mix Cell	Aerated Cell(s)	Total
1. CBOD reduction as a % of average daily design BOD loading.						
2. CBOD removed - Kg/d						
3. CBOD removed - Kg/hr.						
4. AOR - Kg/hr.						
5. ϕ- oxygen transfer correction ratio.						
6. SOR - Kg/hr.						

The highest total oxygen transfer rate is used to size the aeration system for the applied CBOD loading.

7.3.3 Odour Control

7.3.3.1 Odour Production

1. Odour Development

Wastewater contains numerous potentially odourous substances, but the predominant group are the reduced sulphur compounds. Of these, hydrogen sulphide is perhaps the most common and the most easily identified. For this reason, odour control measures concentrate on sulphide control.

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

2. Odour Measurement and Limits

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

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

7.3.3.2 Potential Odour Sources

Odour problems tend to develop when dissolved sulphide concentrations exceed 0.5 mg/L, or less if pH is depressed. Sulphide production commences in the collection system, and will continue to occur wherever anaerobic deposits accumulate. The rate of sulphide production and odour generation are both temperature dependent.

Industrial discharges frequently exacerbate odour. Some discharges have high sulphide contents, others may have a low pH or high temperature.

Turbulence promotes sulphide stripping and hence odours. In the collection system, this occurs at drop manholes, sharp bends, forcemain discharge points and any hydraulic structure where turbulence or super-critical flow develops.

Generally, the odour-emission potential at treatment plants decreases at each successive treatment stage. The influent sewer and headworks receive sewage with higher sulphide content and are often turbulent areas. Preliminary treatment processes can generate odours from the screenings and grit handling areas. Aerated grit tanks will also strip sulphides.

Further sulphide generation often occurs as a result of anaerobic action in the sludge blankets accumulating in primary sedimentation tanks. The resultant hydrogen sulphide can be stripped at the effluent weirs due to the turbulence developed there.

Aeration basins do not usually generate high sulphide odours unless overloaded, as sulphides are oxidized within the basin. Attached growth systems are more likely to generate odours, particularly if the growth becomes excessive. Final clarifiers rarely produce significant odours unless there are problems with the sludge or scum handling systems.

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

7.3.3.3 Evaluation of Odour Production Potential

1. Monitoring Protocols

Detailed monitoring exercises should be preceded by a preliminary study to analyze available data and odour complaints. Complaints should be correlated with data on plant operations and wastewater characteristics and meteorological data. The preliminary study may include limited on-site sampling and analyses.

Detailed field monitoring programs should be of sufficient duration to monitor seasonal variations in sulphide generation and Hydrogen Sulphide emissions. Monitoring should also include an hourly sampling and testing regime to identify typical diurnal fluctuations.

Sampling points should be readily identifiable and remain consistent throughout the monitoring program.

i) Liquid Phase Analyses

Routine parameters to be monitored should include total and dissolved sulphides, CBOD or COD, temperature, pH and dissolved oxygen (D.O.). Oxidation-reduction potential (ORP) can also yield useful data.

Additional analyses for TSS and particle size distribution will be required if the results are to be used for predictive modeling of sulphide generation.

ii) Gas Phase Analyses

In-situ gas phase testing can be used to identify a wide range of odourants, including Hydrogen Sulphide, Mercaptans and Dimethyl Disulphide. Continuous monitoring may be necessary in some cases to identify the peak Hydrogen Sulphide gas concentrations which trigger odour complaints. The designer should ensure that the equipment to be used for gas phase testing is suitable for the range of concentrations expected.

iii) Air Sampling

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

iv) Gas Chromatography

GC analyses are useful for identifying total levels of sulphides, and other potential odourants. Analyses can be carried out on liquid or gas phase samples. The collection and preservation of samples shall be in accordance with established procedures for these type of analyses.

2. Interpretation of Results

Data obtained from monitoring programs will provide the designer with useful information concerning the conditions governing sulphide generation and Hydrogen Sulphide release.

Areas of anaerobic activity producing sulphides will be characterized by low D.O. (less than 0.5 mg/L) and negative ORP. Reaction kinetics will be temperature dependent. The rate of sulphide generation will be greater in the presence of higher fractions of soluble BOD.

Hydrogen sulphide emissions will increase at lower pH and higher temperatures.

3. Predictive Modeling

i) Sulphide Generation in Sewers and Forcemains:

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

ii) Air Quality Computer Modeling

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

7.3.3.4 Odour Control and Abatement Measures

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

1. Prevention of Sulphide Formation

For new treatment facilities, the design should attempt to eliminate ‘dead zones’ where solids may accumulate.

Fillets should be incorporated into rectangular channels. conduits and tanks. Inverted siphons should be avoided.

Where self cleaning velocities cannot be achieved over the full flow range, aeration should be provided to channels and conduits. Excessive aeration should be avoided because of potential odour generation due to increased turbulence. Provide sufficient energy input per unit volume to ensure solids are maintained in suspension.

The designer should also consider improved access provisions to facilitate routine housekeeping and cleaning activities.

2. Chemical Treatment

When dosing chemicals into sewage, side reactions will occur in addition to the desired reaction. In calculating dosing rates, the designer should allow a generous factor of safety to account for these side reactions. Pilot testing is recommended for all chemical dosing systems.

i) Oxidizing Agents

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

ii) Precipitants

Iron and zinc salts will precipitate sulphides. Ferrous and Ferric Chloride have been used extensively, in collection systems, forcemains and at wastewater treatment plants. The designer should consider the effect on the solids handling streams at the wastewater treatment plant, in terms of increased sludge production and increased levels of contaminants in the sludge.

iii) pH Control

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

iv) Electron Acceptors

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

3. Control of Mass Transfer

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

i) Minimize elevation differences where streams converge;

ii) Introduce sidestreams below the liquid surface;

iii) Use of submerged effluent weirs and downstream flow control in lieu of conventional launders for sedimentation tanks and clarifiers.

iv) Avoid excessive or unnecessary aeration.

v) Avoid the use of screw lift pumps on potentially odourous streams.

4. Foul Air Collection and Treatment

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

i) Covers

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

The design of covers should be aimed at minimizing the volume of air requiring treatment.

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

ii) Ventilation

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

iii) Treatment Systems

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

a) Chemical scrubbers, packed bed or mist contactor types;

b) Activated carbon, with or without chemical impregnation;

c) Activated alumina impregnated with potassium permanganate;

d) Biofilter, in-vessel or soil/compost types;

e) Incineration;

f) Dual stage systems comprising one or more of the above.

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

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

Provision will also be required for disposal of waste and side-streams from the treatment systems, which may require further treatment before return to the main liquid stream at the wastewater treatment plant.

An alternative form of biological treatment or pre-treatment involves blowing foul air through aeration basins or trickling filters. If this form of treatment is considered, the designer should be aware of the potential for corrosion in the air blowers and aeration system.

7.3.4 Instrumentation and Controls

7.3.4.1 General

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. AEP monitoring requirements vary depending on the type of facility being considered and its location; this will impact on the selection and type of instrumentation being considered. Instrumentation and control requirements will also depend on the size of the plant, and as each treatment process has its own set of conditions to be monitored and controlled there will be different technical requirements to be met. In general, instrumentation and control should provide efficient and safe automatic and manual operation of all plant systems with a minimum of operator effort. Automatic systems should also be provided with manual back-up systems.

The design should have provision for local control systems where parts of the plant may be operated or controlled from a remote location. The local control stations should include provision for preventing the operation of equipment from remote locations.

When making decisions relating to instrumentation and control, the following factors should be considered:

- Plant size and complexity
- Regulatory requirements
- Hours of attended operation
- Potential chemical and energy savings
- Primary element reliability
- Primary element location
- Whether controls should be manual **or** automatic
- The data storage and recording requirements and whether data acquisition should be central or distributed

For effective operation of larger wastewater treatment facilities the following parameters should be measured, some may not be required for smaller facilities.

- Flow rate for raw sewage, by-pass flows, final effluent flow
- Return Activated Sludge (RAS) flows, Waste Activated Sludge (WAS) flows
- Raw and digested sludge flow, digester supernatant flows
- Chemical dosage, digester gas production

- ! Hazardous gas monitoring
- ! Anaerobic digester temperature
- ! Dissolved oxygen levels
- ! Sludge blanket levels and sludge concentrations

7.3.4.2 Types of Instruments

The different types of instruments that may be required to measure the previously mentioned parameters are classified as primary element devices which alter a signal from a physical process to make it suitable for use by a transmitter. These devices are broken down into function groups with a brief description of the process application.

1. Flow Measurement

(i) Magnetic Flowmeters (Mag Meters)

Liner and Electrode Materials. The liner for the meter can vary depending on the application being considered. In applications where moderate amounts of abrasion are likely to occur, one of the following materials may be selected; Polyurethane, Butyl rubber, Neoprene or Polytetrafluoroethylene. In applications where corrosion is likely to occur, one of the following materials may be selected; Ceramic or Polytetrafluoroethylene. Stainless steel electrode material should be used for applications where corrosion is not likely to present a problem. Hastelloy electrode material should be used for applications where corrosion is likely to present a problem.

Installation. Installation of magnetic flow meters generally require five straight pipe diameters upstream of the meter and three down stream of the meter free of valves or fittings. Meters may be installed on horizontal, vertical or sloping lines. It is essential to keep the electrodes in the horizontal plane to assure uninterrupted contact with the fluid or slurry being metered. The operating velocity required for these meters will fall into the range or (1-10 m/s) for non solids bearing liquids and (1.5-7.5 m/s) for solids bearing liquids. When used to meter liquids containing solids, a continuous electrode cleaner or clean out tee should be installed.

Applications. These meters are suitable for Influent Wastewater, Primary Sludge, RAS, WAS, Digested Sludge and Final Effluent These meters should not be used for Digester Gas or liquid streams with a solids content greater than 10% by weight.

(ii) Ultrasonic Flowmeters

Flowmeter Construction. The flowmeter usually consists of an electronics housing, transducers and pipe section. These can in many cases be fitted to existing pipes either by drilling holes for the transducer hardware or by application of external transducers to the outside of the pipe. When installed on existing pipes, the existing pipe material should be checked to assure it will not dampen the sonic signal as this will adversely affect performance.

Installation. The installation of Ultrasonic flow meters generally require ten to twenty straight pipe diameters upstream of the meter and five down stream of the meter free of valves or fittings. Meters can be installed on horizontal, vertical or sloping lines as long as the pipe sections are always full. The operating velocity required for these meters will fall into the range of (1-10 m/s).

Applications. Transmittance style are not recommended for influent wastewater, primary sludge, thickened sludge, nitrification RAS, or nitrification WAS. Reflective style are not recommended for primary effluent, secondary clarifier effluent final effluent or process wash water.

(iii) Turbine Flowmeters

Flowmeter Construction. The flowmeter usually consists of meter body with rotor blades and a magnetic pickup. The pickup is often connected to electronic display units or a totalizer.

Installation. Installation of turbine flow meters generally require a minimum of ten straight and as high as fifty pipe diameters upstream of the meter and five down stream of the meter free of valves or fittings. Meters may be installed on horizontal or vertical pipelines.

Applications. Turbine flow meters are recommended for applications involving natural gas, compressed digester gas.

(iv) Flumes and Weirs (Parshall Flume)

Installation. The flume will be affected by upstream channel arrangement and it is recommended that there be at least ten channel widths upstream. The flume must also be installed carefully to make certain that it is level.

Applications. Flumes and weirs are customarily used to measure flows in open channels. The are recommended for applications involving open channel flow measurement.

2. Suspended Solids Measurement (Turbidity)

Installation. Installation details for turbidity analyzers are unique to each manufacturer; the manufacturer's recommendations should be followed.

Applications. Turbidity analyzers are recommended for applications involving suspended solids concentrations less than 100 mg/L.

3. Suspended Solids Measurement (Optical)

Installation. Installation details for optical analyzers are unique to each manufacturer; the manufacturer's recommendations should be followed.

Applications. Optical analyzers are recommended for applications involving solids concentrations from 20-mg/L to 8%. Examples are, RAS, WAS and mixed liquor.

4. Dissolved Oxygen Measurement (Galvanic)

Installation. Installation details for dissolved oxygen analyzers are usually related to the choice of placement of the analyzer in the process fluid. The analyzers generally require fairly frequent maintenance and this should be considered in determining the location for installation.

Applications. Oxygen analyzers are recommended for applications involving oxygen concentrations from 0-20-mg/L.

5. Level Measurement

(i) Sonic Ultrasonic

Installation. The mounting location of the sensor is determined from restrictions established by the manufacturer. Typically the sensor must be mounted a minimum distance above the high liquid level and should be located away from tank walls or other obstructions that may cause false echoes.

Applications. This type of level element may be used in many level and flow applications; it is not recommended in locations where foam is dense and persistent.

(ii) Float

Installation. Float switches are normally located in a stilling well when turbulence is expected.

Applications. Float switches are commonly used for high and low level alarms and for controlling pump starts and stops.

(iii) Capacitance

Installation. The installation practices can vary and the manufactures recommended installation should be used.

Applications. May be used in applications that require continuous level measurement and also as switches for alarms or start/stop control.

6. Pressure Measurement

(i) Bourdon Tubes

Installation. The installation practice should include the use of block and bleed valves.

Applications. May be used in applications that require pressure indication. Pressure range 0-35000 kPa.

(ii) Bellows

Installation. The installation practice should include the use of block and bleed valves.

Applications. May be used in applications that require pressure indication. Pressure range 0-2000 kPa.

(iii) Diaphragms

Installation. The installation practice should include the use of block and bleed valves. Transmitters should be installed according to manufacturer's recommendations. Temperature extremes should be avoided and location should be as close as possible to the process measure site.

Applications. May be used in applications that require pressure indication or transmitter output. Pressure range 0-3500 kPa.

7. Temperature Measurement

(i) Thermocouples

Installation. The thermocouple should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications. Thermocouples are suitable for most temperature measurement applications.

(ii) Resistance Temperature Detector

Installation. The Resistance detector should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications. Resistance detectors are suitable for temperature measurement applications with ranges of 0-300 C.

(iii) Thermistor

Installation. The Thermistor should be selected to assure that the device is appropriate for the temperature range. Installation with a thermowell is advised.

Applications. Thermistors are suitable for temperature measurement applications with ranges of 0-300 C.

(iv) Thermal Bulb

Installation. No special installation requirements.

Applications. Thermal bulbs are suitable for temperature measurement applications with ranges of 0-500 C.

7.3.4.3. Process Controls

1. Lift Stations

Lift stations require simple and dependable instrumentation and control systems. The parameters that should be monitored are level, flow, pressure, temperatures, hazardous gas levels, as well as status and alarm conditions. The monitoring and control requirements will vary for each individual case based on the size, location, and economic considerations.

(i) Level Control

Lift stations vary in size and storage capacity but generally they require similar controls. The level in the wetwell increases to the point where a duty pump will be required to start, a lag and follow pump may be started if the level continues to increase. Pumping continues until a pump stop level is reached at which time the duty pump stops, or a series of stop levels will be reached and the lag and follow pumps stop prior to the duty pump. The pump start/ stop control can be performed using any one of several level elements.

When variable speed pumps are used there are several ways in which the pump can be controlled. These generally centre around control to maintain a level set point in the wetwell. This requires a feedback type of control in which the measured variable (level) is compared to a set point value and the final control element is modulated in order to maintain the set point value. Level control of this nature require reliable analog level measurement if it is to function properly. Regardless of the type of level control selected, the system should include a separate low level lockout and high level alarm.

(ii) Flow Monitoring

The flow metering element should be selected carefully to ensure that there are no obstructions where clogging may occur. Provision should be made so that the flow metering element can be bypassed or isolated for routine maintenance activities. The flow metering device should be connected to either the control system or to a recording and totalizing device or both. This provides for a record of flows out of the lift station. It can also be used to help identify possible problems in the discharge piping or force main. [See also Section 7.2.4.2 (5)].

(iii) Pressure Monitoring

Monitoring of the system discharge pressure can be useful in identifying possible problems in the discharge piping or force main and in monitoring pump performance. The pressure metering device should be connected to either the control system or to a recording device or both.

(iv) Pumps and Motors

The following parameters should be monitored:

- Pump bearing temperature;
- Pump bearing vibration;
- Pump speed for variable speed applications;
- Pump discharge pressure;
- Motor voltage and current;
- Motor hours of operation;
- Motor bearing temperature; and
- Motor windings temperature.

(v) Alarms

Lift stations should be alarmed as outlined in Section 7.2.4.5.

2. Mechanical Bar Screens

Three methods are used to control the operation of mechanical bar screens:

(i) Simple manual start/stop which requires the presence of the operator at the screen in order to start and stop the screen.

(ii) Automatic activation by differential level. This method uses the differential level across the screen to provide the start condition. The screen should run at least one complete screen cycle before stopping. The screen can be called to stop when the differential level is returned to a nil value, the final stop should be controlled using a sensor to determine cycle completion (ie limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

(iii) Automatic activation by timer with differential level as emergency start condition. This method uses the differential level across the screen to provide secondary start condition. The screen should run at least one complete screen cycle before stopping. The stop signal should be controlled using a sensor to determine cycle completion (ie limit switch, proximity sensor, timer). When this method is employed there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

3. Primary Treatment

(i) Raw Sludge Pumping

The raw sludge pumping should be set up to incorporate the following features:

! Automatic or manual selection of duty pump;
! On line sludge density metering for control and monitoring;
! On line sludge flow monitoring and totalization;
! On line adjustable sludge density control;
! Individually selectable hopper pumping controls where required;
! Manual override for automatic controls;
! On line sludge blanket level monitoring and alarming;
! On line sludge pump monitoring and control;
! Sludge density feedback control for variable speed pumping with manual override;
! On line sludge pump speed monitoring and control with manual override; and
! On line monitoring and control of primary tank scraper mechanisms.

(ii) Scum Pumping

The scum pumping should be set up to incorporate the following features:

! Automatic or manual selection of duty pump;
! Manual override for automatic controls;
! On line sludge blanket level monitoring and alarming;
! Automatic controls consisting of high and low scum tank level for starting and stopping scum pumps;
! High scum tank level alarm;

- On line scum pump speed monitoring and control with manual override; and
- Scum tank flushing system for scum tank cleaning

4. Secondary Treatment

(i) Dissolved Oxygen (D.O.) Control

Automatic D.O. control systems should be used to control the rate of air supply to aeration tanks. The following methods may be used:

(a) Closed Loop Control (Feedback Control)

Closed loop control consists of on line dissolved oxygen analyzers providing feedback control to an air flow control device. The dissolved oxygen reading is compared to the dissolved oxygen set point. The resultant error signal is used to increase or decrease the rate of air flow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

(b) Feed Forward Control

Feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given flow rate. This system may utilize on line dissolved oxygen analyzers but these are used for monitoring only and do not provide feed back to the air flow control elements.

Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, air flow control elements.

(ii) Return Activated Sludge Control

The Return Activated Sludge pumping should be set up to incorporate the following features:

- Automatic or manual selection of duty pump;
- Variable speed pumping;
- Return Activated Sludge flow monitoring;
- Feedback control to match pumping rates to flow set points;
- Individual control of sludge return rate from individual final clarifiers;
- Manual override for automatic controls; and
- On line monitoring of Return Sludge flow rate, pump speed and status.

(iii) Waste Activated Sludge Control

The Waste Activated Sludge pumping should be set up to incorporate the following features:

- Automatic or manual selection of duty pumps;
- Variable speed pumping;
- Waste Activated Sludge flow monitoring;
- Feedback control to match pumping rates to flow set points;
- Manual override for automatic controls; and
- On line monitoring of Waste Sludge flow rate, pump speed and status.

(iv) Chemical Control System

Chemical addition consists of a feeder or chemical metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control. More specific chemical dosing may also be based on such things as Return Sludge flow rate. Chemical dosing requirements will vary widely depending on performance requirements and the specific process being utilized.

(v) Disinfection Control Systems (Ultra Violet)

The disinfection of final plant effluent utilizing Ultra Violet Light consists of a feed forward control system. This consists of a series of lamps and or lamp channels that are turned on based on effluent flow of the plant, UV transmittance analyzers may be utilized for monitoring system performance but are not generally employed in feedback control.

5. Control and Monitoring Systems

Control and monitoring systems can be a conventional system with recorders, indicators, switches, push buttons, indicating lights, control panels, etc. or it can be a computerized control system that utilizes various configurations of hardware and software to provide the control required. Computerized systems can be separated into two groups, PLC (Programmable Logic Controller) Systems and Distributed Control Systems.

(i) Conventional Relay Control Systems

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions that control the process.

(ii) PLC Control Systems (Programmable Logic Controllers)

The PLC based system is a multipurpose system with extensive scope for modification. The plant status, alarms, motor starters, meters and analyzers are all wired into input/output (I/O) cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The I/O racks are associated with controllers that are programmed to perform the required process control functions. Changes can generally be made relatively easily by modification of or addition to the PLC controller programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of a Man Machine Interface (MMI). The MMI may be dedicated hardware and software or may come in the form of personal computers utilizing MMI software and connected to the PLC communications system. These systems vary widely in there capabilities and performance. The selection of hardware and software should be done carefully to assure current performance and future supportability and expendability.

7.3.4.4 Design Documents

Complete design documents should be prepared to ensure that construction can be completed correctly and also to properly record the system for future reference. The following are required in the design documents:

- Design and construction standards, specifications and installation details.
- Panel sizing and general arrangements.
- Control system functional requirements.
- Control component and instrument data sheets.
- Operator interface and control hardware and software specifications including input and output (I/O) lists.
- control system programming and packaged system configuration standards, structure and scope.

7.3.4.5 Control System Documentation

The following documents should be provided following completion of the control system:

- Record drawings to show any changes to the design and including any drawings produced during construction.
- Annotated listings of control system programs and packaged system configuration.
- Manufacturer's literature for all control and instrumentation components.
- Final wiring diagrams complete with wire and terminal coding.
- Motor control schematics.
- Instrument loop diagrams.
- Panel wiring and layout details.
- PLC or DCS wiring schematics.
- Instrument calibration sheets.
- Operating instructions.

7.3.4.6 Training

Adequate training should be provided to the plant operating and maintenance staff so that the system can be operated to meet the design criteria.

7.3.5 Emergency Facilities and Component Reliability

7.3.5.1 Back-Up Requirements

The components of wastewater treatment plants should be designed in such a way that equipment breakdown and normal maintenance operations can be accommodated without causing serious deterioration of effluent quality.

This does not mean that the tankage capacity has to be doubled. To achieve this, critical treatment processes should be provided in multiple units so that with any unit out of operation, the hydraulic capacity of the remaining units should be sufficient to handle the peak wastewater design flow. There should also be sufficient flexibility in capability of operation so that the normal flow into a unit out of operation can be distributed to all the remaining units.

With some processes such as mechanical screening, the back-up can be provided with a less sophisticated unit such as a manually-cleaned screen.

Sewage and sludge pumping units should be designed such that with any unit out of service, the remaining pumps operating in parallel should be capable of pumping the peak flows. In certain instances, particularly with sludge pumps, one pump may serve as a back-up for more than one set of pumps, i.e. a raw sludge pump could back-up a sludge transfer pump, etc. Standby capacity requirements for sludge return pumps should be determined on a case-by-case basis.

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

- effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);

- time required to perform the necessary repair and maintenance operations;

- the general availability of spare parts and the time required to obtain delivery and installation.

Generally, considerations such as the above will mean that diffused aeration systems will require a standby blower (maximum air demand should be met with the largest blower out-of-service), but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

Chemical feed equipment (e.g. phosphorus removal and disinfection) should be provided in multiple units so that the chemical requirements can be supplied with one unit out of operation.

With sludge digestion facilities, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumpage and mixing so that one stage can be serviced while the other stage receives the raw sludge pumpage. In smaller plants, multiple primary and secondary digestion units can often be avoided by this method. When such an approach is proposed, the designer should outline the alternate methods of treatment and disposal that could be used during periods of equipment breakdown. With larger treatment plants, the provision of multiple primary and secondary digestion units can usually be economically justified. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating.

If effluent filtration is employed, Provision of multiple effluent filtration units may be necessary, depending upon the receiving stream sensitivity, type of filtration equipment, and the maintenance requirements of the filter units.

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

7.3.5.2 Wastewater Bypass Facilities

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

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

Bypass systems should also be constructed so that each unit process can be separately bypassed.

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

7.3.5.3 Standby Power

1. General

Plants should be provided with an alternate source of electric power or pumping capability to allow continuity of operation during power failures, except as noted below. Methods of providing alternate sources include:

(a) The connection of at least two independent power sources such as substations. A power line from each substation should be installed.

(b) Portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy; and

(c) Portable pumping equipment when only emergency pumping is required.

2. Power for Aeration

Standby generating capacity normally is not required for aeration equipment used in the activated sludge process. In cases where a history of long-term (4 hours or more) power outages have occurred, auxiliary power for minimum aeration of the activated sludge may be provided.

3. Power for Disinfection

When receiving water stream is environmentally sensitive, **c**ontinuous disinfection should be provided during all power outages.

7.4 Residual/Biosolids (Sludge) Processing

7.4.1 General

The treatment, handling and disposal of wastewater sludges should be integrated with the planning and design of all wastewater treatment plants. The purpose of sludge processing is to reduce and stabilize biodegradable organic matter so that handling and disposal may be done in an environmentally acceptable manner. Techniques for processing and disposing of sludges will depend on characteristics of the wastewaters and sludges, the wastewater treatment process, and the size and location of the wastewater treatment facility.

Facilities for processing sludge should be provided at all mechanical wastewater treatment plants. Handling equipment should be capable of processing sludge to a form suitable for ultimate disposal. If ultimate disposal method is not suitable year round, provision must be made for sludge storage during the period disposal is not possible.

The selection of sludge handling unit processes should be based upon at least the following considerations:

1. Local land use;

2. System energy requirements;

3. Cost effectiveness of sludge thickening and dewatering;

4. Sludge digestion or stabilization requirements;

5. Sludge storage requirements; and

6. Methods of ultimate disposal.

7.4.2 Biosolids (Sludge) Handling and Treatment

7.4.2.1 Digestion

Sludge stabilization is generally achieved by digestion. Two types of digestion systems are used - anaerobic and aerobic.

Anaerobic digestion is the most commonly used system for the digestion of primary and mixtures of primary and waste activated sludges. Aerobic digestion, because of the relatively large energy requirements, is not recommended for use, particularly at large wastewater treatment facilities.

1. Anaerobic Digestion

Anaerobic sludge digestion is used to reduce and stabilize the biodegradable organic matter to improve the dewatering characteristics of sludge and to reduce pathogenic organisms. Bulky, odorous raw sludges are converted to a relatively inert material that can be readily dewatered in the absence of offensive odours. Oxygen is excluded from the anaerobic digestion process, and most pathogenic organisms are destroyed by properly designed digester tanks operating at temperatures around 35°C over a period of about 15 to 30 days. Temperature, pH, mixing, and retention time are the critical design factors which should be considered.

Anaerobic digesters should meet the requirements of the latest edition of "CAN/CGA-B105, Code for Digester Gas and Landfill Gas Stations. Digestion systems should also be designed with features and in accordance with design parameters, as follows:

- *Number of Stages*

  Two (primary and secondary)

- *Number of Digesters in Each Stage*

  One adequate in small plants, provided that flexibility is provided to allow either stage to receive raw sludge in emergencies; number of digesters in each stage of large plants will be dictated by economics.

- *Hydraulic Retention Time in Primary Digester*

  Minimum 15 days (sludge retention time requirements of slowest methane producers is approximately 10 days).

- *Mixing*

  For digestion systems utilizing two stages, the first stage (primary) should be completely mixed (via digester gas - compressor power requirements 5 to 8 W/m^3 or mechanical means - 6.6 W/m^3). The second stage (secondary) is to be designed for sludge storage, concentration, and gas collection and should not be credited in the calculations for volumes required for sludge digestion.

- *Volatile Solids Loading*

  0.5 - 1.5 $kg/m^3/d$.

- *Completely Mixed Systems*

  For digestion systems providing for intimate and effective mixing of the digester contents, the system may be loaded with volatile solids up to 1.3 kg/m^3 of volume per day in the active digestion units.

- *Moderately Mixed Systems*

  For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded with volatile solids up to 0.65 kg/m^3 of volume per day in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided.

- *Heating*

  Heating must be provided for the primary digester so that a temperature of 35°C can be maintained. External heat exchanger systems are preferred. Heating should be via a dual-fuel boiler system using digester gas and natural gas, or oil.

- *Digester Covers*

  Digester covers may be fixed or floating type, sized to provide gas storage volume. Insulated pressure and vacuum relief values and flame traps should be provided. Access manholes and sampling wells should also be provided in the digesters covers. The underside of the roof must be protected from corrosion. Coatings should be compatible with the roofing material and the environment in the digester to ensure a good bond.

  Steel, concrete or fibreglass covers may be used.

- *Secondary Digester Sizing*

  The secondary digester should be sized to permit solids settling for decanting and solids thickening operations, and in conjunction with possible off-site facilities, to provide the necessary digested sludge storage. Off-site storage in sludge lagoons, sludge storage tanks, or other facilities, may be used to supplement the storage capacity of the secondary digester.

- *Sludge Piping*

  Maximum flexibility should be provided in terms of sludge transfer from primary and secondary treatment units to the digesters, between the primary and secondary digesters, and from the digesters to subsequent sludge handling operations; minimum diameter of sludge pipes should be 100 mm; provision should be made for flushing and cleaning sludge piping; sampling points should be provided on all sludge lines; main sludge transfer lines should be from the bottom of the primary digester to the mid-point of the secondary digester.

- *Supernatant Piping*

  Supernatant should be returned to the treatment plant with flexible points of return to the grit removal facilities, upstream of the primary settling tanks, or to the aeration tank; multiple draw-off points or adjustable supernatant draw-offs, and sampling points should be provided; both primary and secondary digesters should be equipped with supernatant piping so that during emergencies the primary can be operated as a single stage process; additional $CBOD_5$ load caused by supernatant return should be considered in aeration system design.

- *Overflows*

  Each digester should be equipped with an emergency overflow system.

- ***Waste Gas***

  i) Location

  Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

  ii) Pilot Light

  All waste gas burners shall be equipped with automatic ignition such as a pilot light or a device using a photoelectric cell sensor. Consideration should be given to the use of natural or propane gas to insure reliability of the pilot.

  iii) Gas Piping Slope

  Gas piping shall be sloped at a minimum of 2 percent up to the waste gas burner with a condensate trap provided in a location not subject to freezing.

2. Aerobic Digestion

Aerobic digesters treating waste activated sludge should be designed in accordance with the following criteria. If primary sludge is to be included, minimum sludge age and air requirements may have to be increased.

- *Number of Stages*

  Two.

- *Number of Tanks in Each Stage*

  Generally one.

- *Loading*

  1.5 $kg/m^3.d$ volatile suspended solids based upon first stage volume only.

- *Sizing*

  Designed to achieve a minimum sludge age of 45 days, including both stages and sludge age of waste activated sludge; if a total of 45 days sludge age is all that is provided, it is suggested that 2/3 of the total digester volume be in the first stage and 1/3 be in the second stage; if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters.

- *Air and Mixing Requirements*

  Aeration rate will depend upon the oxygen uptake rate at the maximum solids content experienced; as a guideline, 0.85 L/m^3.s (litres of air per cubic metre of aeration tank per second) should be provided for diffused aeration systems; a minimum bottom velocity of 0.25 m/s should be maintained while aerating; mechanical surface aeration systems are not recommended due to increased heat loss causing icing problems.

- *Tank Design*

  Generally open; tankage should be of commonwall construction or earthern-bermed to minimize heat loss; tank depths 3.6-4.6 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths to, or from both the primary and secondary digester.

7.4.2.2 Conditioning

Sludge dewatering, and to a lesser extent sludge thickening operations, are highly dependent upon sludge conditioning for their successful operation. Sludge conditioning not only affects the solids concentration of the thickened or dewatered sludge, but also affects the solids capture efficiency of the process.

There are two sludge conditioning approaches that can be used. Sludge can be conditioned by physical methods, such as heat treatment , or by chemical methods, involving the addition of either organic or inorganic chemicals.

The method selected will not only differ in its effect on the thickening or dewatering process, but will have different effects on subsequent sludges handling operations and on the sewage treatment process itself.

1. Physical Methods

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. With this process, the sludge is treated at temperatures of 175 to 204°C, pressure of 1700 to 2800 kPa and for detention times of 15 to 40 minutes. The high temperatures cause hydrolysis of the water-solids matrix and breaking down of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the water-solids separation characteristics.

Although the heat conditioning system has been proven to be an effective sludge conditioning technique for subsequent dewatering operations, the process results in a significant organic loading to the aeration tanks of the sewage treatment plant, if the supernatant is returned to the aeration system, due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50 percent of the total loading on the aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials such as stainless steel. Scale formation in the heat exchangers, pipes and reactor is a common problem.

The design requirements for a heat conditioning system should be determined by either batch or small-scale continuous pilot-plants. Through such methods, the necessary level of hydrolysis to produce the desired reduction in the specific resistance of the sludge, and the liquor characteristics can be determined. Tests can also be made at different temperatures and retention times to determine the most effective full-scale operating conditions.

Freezing of sludges has been used successfully for water treatment plant sludges, but not common as a conditioning method for sewage sludges.

2. Chemical Methods

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. Chemical conditioning for thickening operations attempts to promote more rapid phase separation, higher solids concentration and a greater degree of solids capture. With dewatering operations, chemical conditioning is used in an attempt to enhance the degree of solids capture by destabilization and agglomeration of fine particles. This promotes the formation of a cake which then becomes the true filter media in the dewatering process.

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are high molecular weight polymers. The selection of the most suitable chemical(s) and the dosage requirements for sludge conditioning can be best determined initially by bench and pilot testing.

Laboratory testing should, however, be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15 percent of full-scale needs.

7.4.2.3 Thickening

Sludge thickening can be employed in the following locations in a wastewater treatment plant:

- prior to digestion for raw primary, excess activated sludge or mixed sludges;

- prior to dewatering facilities;

- following digestion for sludges or supernatant;

- following dewatering facilities for concentration of filtrate, decant, centrate, etc.

The commonly employed methods of sludge thickening are gravity and air flotation. Their suitability for the various types of sludge are shown in Table 7.9. Centrifuges, gravity belt thickeners and rotating drum thickeners are also used for sludge thickening. All thickening devices are adversely affected by high sludge volume indexes (SVIs) and benefited by low SVIs in the influent activated sludges. The ranges of thickened sludge concentrations given in Table 7.9 assume an SVI of approximately 100.

Wherever thickening devices are being installed special consideration should be given to the need for sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines, and thickening equipment. Also, where thickeners are to be housed, adequate ventilation should be provided.

1. Gravity

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone.

Gravity thickeners should be designed in accordance with the following parameters:

- *Tank Shape*

  Circular.

- *Tank Depth*

  3 to 3.7 m.

- *Tank Diameter*

  Up to 21 - 24 m.

- *Floor Slope*

  Acceptable range 2:12 to 3:12.

- *Solids Loading*

  Primary sludges 96 to 120 kg/m^2.d; waste activated 12 to 36 kg/m^2/.d; combination of primary and waste activated based on weighted average of above loading rates.

- *Overflow Rate*

  0.19 to 0.38 L/m^2.s.

- *Chemical Conditioning*

  Provision should be made for the addition of conditioning chemicals into the sludge influent lines.

- *Sludge Volume Ratio*

  Volume of sludge blanket divided by volume of sludge withdrawn daily should be 0.5 to 2 days.

TABLE 7.9
SLUDGE THICKENING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Thickening Method	Sludge Type	Performance Expected
GRAVITY	Raw Primary	Good, 8 to 10% Solids
	Raw Primary and Waste Activated	Poor, 5 to 8% Solids
	Waste Activated	Very Poor, 2 to 3% Solids (Better results reported for oxygen excess activated sludge)
	Digested Primary	Very Good, 8 to 14% Solids
DISSOLVED AIR FLOTATION	Waste Activated (Not generally used for other sludge types)	Good, 4 to 6% Solids and $>$ 95% Solids Capture

2. Air Flotation

Unlike heavy sludges, such as primary and mixtures of primary and excess activated sludges, which are generally most effectively thickened in gravity thickeners, light excess activated sludges can be successfully thickened by flotation.

Flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters and some bench-scale and/or pilot-scale testing will be necessary. The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operation:

- *Tank Dimensions*

  Vary with suppliers.

- *Air Buoyancy Systems*

  Vary with suppliers.

- *Air to Solids Weight Ratio*

  0.02 to 0.05.

- *Recycle Ratios*

  Vary with suppliers (0 to 500%).

- *Solids Loadings (with waste activated sludge to achieve 5% float solids)*

  48 $kg/m^2.d$ (without flocculating chemicals); up to 240 $kg/m^2.d$ (with flocculating chemicals).

- *Chemical Conditioning*

  Feed chemical to mixing zone of sludge and recycled flow.

- *Hydraulic Feed*

  Up to 1.74 $L/m^2/.s$ (based on total flow including recycle, when polymers used); without chemicals, lower rate must be used; feed rate should be continuous.

- *Detention Time*

  Not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

7.4.2.4 Dewatering

Sludge dewatering will generally be required prior to ultimate disposal of sludges, other than for land application. Since the processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major influence on the dewatering method most suitable for a particular wastewater treatment plant. Also of influence will be the characteristics of the sludge requiring dewatering, that is, whether the sludge is raw or digested, whether the sludge contains waste activated sludge, or whether the sludge has been previously thickened.

Table 7.10 gives the solids capture, solids concentrations normally achieved, energy requirements and suitable ultimate disposal options for various dewatering methods. The solids concentrations shown in Table 7.10 assume that the sludges have been properly conditioned.

In Alberta, the requirement for landfilling municipal sludges is dictated by Public Health Regulations. In general, the solids concentration for sludges which are to be landfilled at sanitary landfill sites will be influenced by the quantities of sludge to be disposed of in relation to the quantities of municipal refuse, characteristics of the site itself, and the expected effects of the liquid addition to the site. With small quantities of sludge for co-disposal landfilling with garbage, liquid sludge at solids concentrations as low as 3 percent may be acceptable. For sludge only landfill operations, a minimum of 15 percent solids concentration is generally required to support cover material.

If sludge is to be disposed of in sludge lagoons, dewatering may not be necessary unless it is justifiable for economic reasons relating to haulage costs.

In Alberta, the most prevalent method of sludge disposal is by land application on agricultural lands. Due to the fact that the ammonium nitrogen content of sludges is largely associated with the liquid fraction of sewage sludges and the acceptability of sludges for spreading on agricultural land relates to minimum ratios of nitrogen to heavy metal concentrations, dewatered sludges will generally be less desirable for final spreading on agricultural lands than liquid sludges. To enable sludges to be handled and spread as liquids, the upper limit for solids content will generally be in the order of 12 percent. This would leave only thickened sludges acceptable for spreading on agricultural land by liquid spreading techniques, and sludge dewatering will not be necessary. For this reason, no design guidelines have been included in this document for sludge dewatering facilities. However, should it become necessary to dewater sludges, the proponent should consult with AEP before proceeding with the design of the facility.

TABLE 7.10
SLUDGE DEWATERING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Dewatering Method	Solids Capture (%)	Solids Concentrations Normally Achieved	Median Energy Required (m^3/dry tonne)	Suitable Ultimate Disposal Methods	
				Landfill	Agricultural Utilization
Filter Press	90 to 95	Raw Primary + WAS - (30 to 50%) Digested Primary + WAS - (35 to 50%) WAS - (25 to 50%)	360	Yes Yes Yes	No No No
Centrifuge (Solid Bowl)	95 to 99	Raw or Digested Primary + WAS - (15 to 25%) WAS - (12 to 15%)	360	Yes Yes	No No
Belt Filter	85 to 95	Raw or Digested Primary + WAS - (14 to 25%) WAS - (10 to 15%)	130	Yes Yes	No No

8.0 STORMWATER MANAGEMENT GUIDELINES

8.1 General

This section provides a brief summary of the design standards and guidelines for storm drainage systems in Alberta. Detailed stormwater management standards and guidelines are described in the AEP publication entitled, Stormwater Management Guidelines for the Province of Alberta.

8.2 Stormwater Collection

8.2.1 Dual Drainage Concept

Dual drainage concept (minor and major systems) should be followed in the design of the collection systems. The minor system (underground pipe systems, roof leaders, gutters, lot drainage, etc.) provides a basic level of service by conveying flows during minor storm events; the major system (lot drainage, roads and gutters, storage facilities, etc.) conveys runoff from the extreme events in excess of the minor system capacity.

There is always a major system, whether or not one is planned. Failure to plan for a major system often results in unnecessary flood damage.

8.2.1.1 Design Capacity

The establishment of capacity criteria for the minor system is largely a trade-off between cost and convenience in terms of level of service. For larger municipalities, the minor system should be designed to carry the peak flow resulting from a one in 5 year rainfall event; for several communities faced with limited financial reserves, the use of the 2 year event may be practical.

For the major system, the design should be based on a one in 100 year rainfall event.

8.2.2 Storm Sewers

Storm sewers shall be designed as a separate sewer system. Effluent from sanitary sewers or any potentially contaminated drainage from industrial, agricultural, or commercial operations shall not be discharged to storm sewers.

Contaminated drainage means, the introduction of any foreign, undesirable physical, chemical or biological substance into the environment which results or is likely to result in deleterious effects.

8.2.2.1 Sewer Hydraulics

Storm sewer pipe shall be designed to convey the design flow when flowing full with the hydraulic gradeline at the pipe crown. Crown elevations should match at manhole junctions.

8.2.2.2 Flow Velocities and Minimum Slope

Storm sewer flow velocities shall not be less than 0.60 m/s when flowing full. refer to Table 8.1 for minimum slopes of gravity storm sewers.

If sewer flow velocities exceed 3 m/s, special consideration shall be given to prevent scouring.

TABLE 8.1

MINIMUM DESIGN SLOPES FOR STORM SEWERS

Sewer Diameter (mm)	Minimum Design Slope (m/100m)
300	.194
375	.145
450	.114
525	.092
600	.077
675	.065
750	.057
900	.045
1050	.036
1200	.031
1350	.027
1500	.023
1650	.020
1800	.018
1950	.016
2100	.015
2250	.013
2400	.012
2550	.011
2820	.010

Note: Design slopes based on a minimum velocity of 0.60 m/s for pipe flowing at least half full

8.2.2.3 Pipe Size

The minimum diameter for storm sewers shall be 300 mm.
The minimum diameter for catch basin leads shall be 250 mm.

8.2.2.4 Pipe Material

The selection of pipe material, pipe classes and bedding types should be based on loading conditions. The designer should be particularly careful to specify sulphate resistant concrete pipe in areas of sulphate soil.

Storm sewer pipe shall have been manufactured in conformity with the latest standards by the American Society for Testing Materials (ASTM) or the Canadian Standards Association (CSA).

8.2.2.5 Pipe Cover

The minimum depth of cover to pipe crown shall be 1.2 m.

8.2.2.6 Curved Sewers

Curved sewers shall match the roadway curvature by means of deflection at the joints only. Joint deflections shall not exceed the manufacturer's specified allowable deflection. Consideration should also be given to increasing the grade of curved sewers to offset increased head loss.

8.2.2.7 Change in Flow Direction

For storm sewer pipes greater than 600 mm in diameter, changes in flow direction at manholes should not exceed 45^{o}. This limitation may be exceeded if care is taken to design a proper transition manhole.

8.2.2.8 Extraneous Flows

Roof leaders shall not be connected to storm sewers in residential areas, but shall discharge to grassed or pervious areas. Roof leaders from multi-family, commercial, and industrial sites and foundation drains may be connected to storm sewers at the discretion of the Local Authority.

8.2.2.9 Sewer Maintenance

Control should be provided to minimize sediment discharge to storm sewers. This control may be in the form of properly graded and surfaced streets and lanes, landscaping, catch basin sumps, or sediment control structures at pond and lake inlets.

8.2.3 Storm Manholes

The design of storm manholes should conform in all respects to section 7.2.2 pertaining to the design of sanitary sewer manholes, with the following exception:

For storm sewers, 1.0 m in diameter or larger, a bend may be installed instead of a manhole at all changes in grade or alignment.

8.2.4 Catch Basins and Gutters

8.2.4.1 Collection of Surface Runoff

Surface water should not be permitted to run a distance greater than 300 m along roadways without interception by the first catch basin. From this first point of interception, surface runoff should not run a distance greater than 120 m between catch basins.

8.2.4.2 Catch Basin Capacity

The inlet capacity of each catch basin should be sufficient to receive the calculated surface stormwater flow at that location. The minimum inside diameter of catch basin leads shall be 250 mm.

8.2.4.3 Catch Basin Construction

All catch basin bodies shall be of either 600 mm or 900 mm precast concrete sections. Where a sump cleaning maintenance program is in effect, the body shall be constructed so as to provide a 600 mm sump to trap silt and gravel.

8.2.4.4 Gutters

The minimum grade of gutters used to intercept stormwater runoff should be 0.40%. Gutters of less than 20 m in length or curved gutters of short radius should have a minimum grade of 0.60%.

8.2.5 Stormwater Pumping Stations

Being that stormwater pumping is an uncommon practice, there are no specific criteria in these standards with respect to the design and operation of stormwater pumping stations. The proponent of a pumping station should contact the Director of Air and Water Approvals before commencing with detailed design.

8.3 Stormwater Best Management Practices (BMPs)

8.3.1 Introduction

8.3.1.1 General

Stormwater Best Management Practices (BMPs) are methods of managing stormwater drainage for adequate conveyance and flood control and are economically acceptable to the community. BMPs are stormwater management methods that retain as much of the “natural” runoff characteristics and infiltration components of the undeveloped system as possible and reduce or prevent water quality degradation.

Stormwater BMPs that may be considered for stormwater quantity and quality controls are discussed in the following order:

- Source control BMPs
- Lot-level BMPs
- Conveyance system BMPs
- End-of-pipe BMPs

8.3.1.2 Design Criteria for Stormwater Quality Control

It is considered that storing the volume of runoff from a 25-mm storm over the contributing area is appropriate for Alberta for stormwater quality control using detention devices such as dry ponds, wet ponds, and constructed wetlands. A detention time of 24 hours should also be used for detention facilities. The runoff from a 12-mm storm over the contributing area is considered appropriate for infiltration BMP's.

8.3.2 Source Control BMPs

Removal of stormwater contaminants at their source may, in some instances, be a practical solution to the mitigation of pollutant impacts. There are three main pollutant removal activities that are normally practiced by a municipality for source control including street sweeping, catchbasin cleaning, and animal litter removal through enforcement of bylaws.

8.3.3 Lot-Level BMPs

Stormwater lot-level controls are practices that reduce runoff volumes and/or treat stormwater before it reaches a subdivision/development conveyance system. This type of controls can be readily incorporated into the design of future developments. With all development, the applicability of stormwater lot-level controls should be investigated before conveyance and end-of-pipe systems are examined.

Traditional lot-level controls aimed at stormwater quantity management and the reduction of peak runoff rates include:

- Restricting numbers of roof drains to provide rooftop detention of stormwater

- Installing catchbasin restrictors or orifices in the storm sewer to promote parking lot detention

- Oversizing storm sewers and installing orifices in the sewer to create pipe storage

- Installing catchbasin restrictors in rear yard catchbasins to create rear yard storage

The above-noted lot-level measures are primarily designed to reduce runoff peaks. Other stormwater management criteria, such as the preservation of water quality, protection from erosion, and the maintenance of baseflow are not adequately addressed through these techniques. Lot-level controls that help preserve the natural hydrologic regime include:

- Reduced lot grading
- Directing roof leaders to rear yard ponding or soakaway pits

- Sump pumping foundation drains to rear yard ponding areas

8.3.3.1 Reduced Lot Grading

1. Purpose

The purpose of reducing lot grades is to reduce the volume of runoff from developed lots by increasing the travel time of runoff, and increasing the availability and opportunity for depression storage and infiltration. A significant reduction in lot-level runoff volumes would also affect the other minor stormwater system components and the major system components by reducing the conveyance and treatment requirements.

2. Description

Typical development standards require a minimum lot grade of 2 percent to drain stormwater away from buildings. In flat areas, a reduction to minimum lot grades should be evaluated. In hilly areas, alterations to natural topography should be minimized. To avoid foundation drainage problems, the grading within 2 to 4 m of buildings should be maintained at 2 percent or higher. Areas outside of this envelope should be graded at less than 2 percent.

Reduced lot grading BMPs promote depression storage and natural infiltration and reduces risks associated with flooding and erosion. The maintenance of natural infiltration could have positive impacts on baseflow depending on local evapotranspiration rates.

3. Applicability

Reduced lot grades can be recommended as a lot-level stormwater BMP for any new developments and in regrading or re-landscaping of existing lots in established developments.

4. Effectiveness

Very little information is available in regard to the impact that reductions in lot grades may have on the overall runoff volumes from a developed area. It has been recommended that reductions in lot grading may increase the pervious depression storage by as much as 1.5 mm for a 0.5 percent to 2.0 percent change in grade. Reduction of on-lot runoff will also reduce downstream erosion potential.

5. Water Quantity

Reduced lot gradings limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater also provides recharge to the local groundwater that may, in turn, discharge to local streams thus enhancing baseflows.

6. Water Quality

Reduced lot gradings limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. The effectiveness of reduced lot grades in limiting contaminant runoff is also dependent on land use.

7. Design Considerations

Design guidelines for on-lot grade reductions are shown in Figure 8-1. Grades within 4 m of structures should be maintained at 2 percent. Grades beyond 4 m of structures should be reduced to 0.5 percent. Consideration should also be given to tilling soils in flatter grade areas to a depth of 30 mm prior to seeding or sodding to reduce soil compaction and increase infiltration potential.

8.3.3.2 Surface Ponding and Rooftop Storage

1. Purpose

Roof leaders that discharge to surface ponding areas reduce the potential for downstream flooding and erosion and help maintain pre-development end-of-pipe discharge rates. The same benefits can result from the use of rooftop storage, which are likely suitable for commercial, industrial, and institutional buildings.

2. Description

Roof leaders are directed toward rear lot depressions that allow stormwater to infiltrate or evaporate. For rooftop storage roof, drains on flat roofs are raised to allow ponding on the rooftop.

3. Applicability

Surface ponding areas can be recommended as a lot-level stormwater BMP for any new developments and in regrading or re-landscaping of existing lots in established developments. Surface ponding may also be used for parking lots or park areas. Rooftop storage can be recommended for industrial, commercial, or institutional buildings with flat roofs.

4. Effectiveness

Rear lot ponding of stormwater or rooftop storage effectively limits runoff by a volume equal to the amount of impervious depression storage provided.

5. Water Quantity

Rear lot ponding and rooftop storage limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing baseflows.

6. Water Quality

Rear lot ponding and rooftop storage limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

7. Design Considerations

Design guidelines for rear lot ponds are shown in Figure 8-2. Maximum depths should be maintained at 100 mm. Flow paths should be provided to direct overland flow to the pond. To maintain the pond, catchbasins can be elevated to the required height or grassed swales can be created. More complex designs may incorporate an infiltration trench beneath the ponded area to enhance infiltration. The pond should be sized to accommodate a minimum of 5 mm and a maximum of 20 mm of rainfall covering the roof area. Rooftop ponding can be accomplished by raising roof hoppers to create a maximum ponding depth of 10 mm.

8.3.3.3 On-lot Infiltration Systems

1. Purpose

On-lot infiltration systems are used for detention of stormwater from relatively small catchment areas. Infiltration systems may be used in areas without adequate minor system conveyance. They also provide enhancement to water quality and reductions in overland flow.

2. Description

Infiltration systems may be simply designed pits with a filter liner and rock drain material or more complex systems with catchbasin sumps and inspection wells. Stormwater flow from roof drains is directed to the infiltration system.

3. Applicability

Infiltration systems are recommended for relatively small detention volumes. If larger detention volumes are required a series of infiltration basins may be employed. Infiltration basins should not be built under parking lots or other multi-use areas, if the groundwater table is within 0.6 m of the infiltrating surface, if bedrock is located within 1.2 m of the infiltration surface, if the infiltrating surface is located on top of fill material and if the underlying soils have a fully saturated percolation rate of less than 1.3 mm.

4. Effectiveness

Infiltration systems have a number of advantages over rear yard ponding including increased groundwater recharge and less inconvenience to home owners. Infiltration systems may have increased maintenance requirements over ponds and a more uncertain operating life. On-lot infiltration systems accept only roof runoff and are therefore subjected to minimal levels of suspended solids.

5. Water Quantity

On-lot infiltration systems limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing baseflows.

6. Water Quality

On-lot infiltration systems limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

7. Design Considerations

Figures 8-3 and 8-4 illustrate two different applications of infiltration systems. The total void volume should be calculated from the storage required for the 2 year design storm which is calculated from the effective porosity of the infiltration fill material. The infiltration surface area required (bottom surface area) to drain the system within 48 hours is calculated from the 24-hour sustained percolation rate. An overland flow path should be provided for overflow volumes during saturated or frozen conditions. A pretreatment filter (Figure 8-3) or sump (Figure 8-4) should be provided to limit solids input into the system. Design void space volumes are calculated from the volume of water required to fill a known volume of drain rock. A suitable quality filter fabric or geotextile must also be incorporated into the design.

In locating infiltration systems, consideration should be given to proximity to septic fields.

8.3.3.4 Sump Pumping of Foundation Drains

1. Purpose

Many current development standards allow foundation drains to be directly connected to the storm sewer. By pumping foundation drainage to surface or subsurface ponding/soakaway areas, infiltration, flooding, and erosion water management concerns may be reduced.

2. Description

Foundation drainage is sometimes pumped to the storm sewer network, to a suitable infiltration system, or to the surface where it is conveyed to a catchbasin and then to a storm sewer.

3. Applicability

Sump pumps are not feasible in areas where the seasonal high groundwater table is within 1 m of the foundation drain. Sump pumps are not feasible in areas where bedrock is within 1 m of the foundation drain. Application under these conditions may cause excess pumping. Under other conditions where infiltration systems are appropriate or where overland flow paths are available sump pumps can be recommended to discharge to either the infiltration system or to the surface.

4. Effectiveness

Foundation drainage is normally relatively clean water and is well suited to the optimal operation of infiltration systems or overland flow to rear yard ponds.

5. Water Quantity

The impact of foundation drain discharge on downstream stormwater management facilities is dependent on the original discharge location. If foundation drainage was originally discharged to the storm sewer network or to the sanitary sewer, there will be some reduction in stormwater flow in the sewer. There will also be additional groundwater recharge and potentially baseflow augmentation in the local receiving stream if foundation drainage was originally discharged to either the storm sewer or sanitary sewer networks.

6. Water Quality

Foundation drainage is relatively clean water and if flow is removed from either the storm sewer network or the sanitary sewer network there is likely to be some impact on the dilution of contaminants provided by the foundation drainage.

7. Design Considerations

Sump pump drainage to an infiltration system is illustrated in Figure 8-5. The location of the infiltration system should conform to infiltration design considerations. Yard grades should conform to design considerations for infiltration ponds. Sump pump discharges should be located at least 2.0 m away from foundations and be discharged to rear yards away from sidewalks to prevent icing conditions during winter months. Discharges should also be located at least 0.5 m above ground to prevent blockage from ice and snow during the winter.

8.3.4 Stormwater Conveyance System BMPs

Stormwater conveyance systems transport drainage from developed areas through sewer or grassed swale systems. Stormwater conveyance controls are applied as part of the stormwater conveyance system and can be classified into three categories:

- Pervious pipe systems
- Pervious catchbasins
- Grassed swales

8.3.4.1 Pervious Pipe Systems

1. Purpose

Pervious pipe systems are intended to convey and infiltrate road drainage.

2. Description

Pervious pipe systems are perforated along their length, thereby promoting exfiltration of stormwater as it is conveyed downstream. The system is very similar to a conventional tile drainage system.

Pervious pipe networks are components of roadway drainage systems. Because roadway drainage usually carries a high level of suspended sediments there are associated pretreatment components. Roadway runoff is normally directed toward grassed areas that act as sediment filters prior to flowing into the stormwater catchbasin. The stormwater catchbasin is raised to allow some ponding and further sediment removal. The catchbasin is connected to the pervious pipe.

3. Applicability

Pervious pipe systems, although being implemented in several municipalities, are still considered experimental in nature.

4. Effectiveness

Pervious pipe systems for the exfiltration of road runoff have not proven very reliable. Pervious pipe systems experience clogging due to the high solids loads especially during construction of the pervious pipe system in new developments.

5. Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally quite impervious. Any stormwater infiltrated through the pervious pipe network reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6. Water Quality

Road runoff normally carries high levels of solids, oils, greases, metals, and chlorides if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

Infiltration of road runoff may, however, present a groundwater contamination problem.

7. Design Considerations

Implementation of a pervious pipe system is illustrated in Figure 8-6. Design considerations must include the pretreatment of road runoff for solids removal. Pretreatment can be accomplished by incorporating grassed boulevards as pretreatment areas. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. The infiltration pipe must be a sufficient height above the groundwater table to prevent groundwater from flowing into the pipe and allow for proper infiltration.

The minimum storage volume should be equal to the runoff from a 5-mm storm over the contributing drainage area. The storm volume should be accommodated in the pervious pipe bedding/storage media without overflowing. The maximum storage area should be equal to the runoff from a 25-mm storm over the contributing drainage area. The exfiltration storage bedding depth should be 75 mm to 150 mm deep above the crown of the pervious pipe and the bedding should drain 24 hours. The minimum diameter for the pervious pipe should be 200 mm and the pipe should be smooth walled to reduce the potential for clogging

8.3.4.2 Pervious Catchbasins

1. Purpose

Pervious catchbasins are intended to convey and infiltrate road drainage.

2. Description

Pervious catchbasins are normal catchbasins with larger sumps that are physically connected to an exfiltration storage media. The storage media is generally located beneath or beside the catchbasin.

3. Applicability

Pervious catchbasins are still considered to be experimental.

4. Effectiveness

Maintenance requirements for pervious catchbasins are dependent on the clogging frequency of the infiltration media which can be high given the sediment load normally associated with road runoff. Pervious catchbasins are easier to construct in new developments because they can be plugged during construction to prevent solids clogging the system.

5. Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally quite impervious. Any stormwater infiltrated through pervious catchbasins reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6. Water Quality

Road runoff normally carries high levels of solids, oils, greases, and metals. Chlorides may also be a problem if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

7. Design Considerations

The application of a pervious catchbasin for road runoff control is illustrated in Figure 8-7. The most important design consideration is the provision of adequate pretreatment of solids to prevent frequent clogging. Design specifications recommend construction at least 1 m above the groundwater table and the use of appropriate unwoven geotextile and clear 50-mm stone to promote filtration with a low clogging frequency. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. Storage volume criteria should be the same as that for pervious pipe. The depth of the exfiltration storage is dependent upon the native soil characteristics. Maximum depths can be calculated based on the native soil percolation rate. The physical dimensions of the storage will depend on the area of land available.

8.3.4.3 Grassed Swales

1. Purpose

Grassed swales store, infiltrate and convey road and on-lot stormwater runoff. Grassed swales are normally associated with more rural low-density developed drainage basins.

2. Description

Grassed swales are natural depressions or wide shallow ditches. The grass or emergent vegetation in the swale acts to reduce flow velocities, prevent erosion, and filter stormwater contaminants.

3. Applicability

Grassed swales are typically used in more rural areas with rolling or relatively flat land but can be used in place of or as an enhancement to any stormwater curb and gutter system except in strip commercial and high-density residential areas. In rural areas and in urban applications, grassed swales have been shown to effectively infiltrate runoff and remove pollutants. Grassed swales are being designed more frequently to replace curb and gutter controls and can be recommended for consideration in both rural and urban drainage basins.

4. Effectiveness

Grassed swales have been reported to provide effective quantity and quality control of urban and rural runoff. Grassed swales must be properly maintained to ensure effectiveness and prevent ponding of water. If water is allowed to pond in the swale, wetland vegetation may grow and mosquitos may become a problem.

5. Water Quantity

Grassed swales infiltrate stormwater and reduce the end-of-pipe discharge volumes normally associated with curb and gutter controls. Significant amounts (up to 95 percent) of runoff reduction are reported in the literature pertaining to grassed swales. Grassed swales also significantly lower peak discharge rates associated with frequent storms. The changes in runoff discharge volumes and rates also reduce erosion in downstream systems.

6. Water Quality

Grassed swales can be effective in filtering and detaining stormwater runoff from a variety of catchment types. Grassed swales are effective for stormwater treatment as long as minimum channel slope is maintained and a wide bottom width is provided. Many stormwater contaminant particulates are effectively filtered by grassed swales including heavy metals, COD, nitrate nitrogen, ammonia nitrogen, and suspended solids. Other contaminant nutrients such as organic nitrogen, phosphorus, and bacteria have been reported to bypass grass swales.

7. Design Considerations

General design considerations for a grassed swale are shown in Figure 8-8. An illustration of a grassed swale with a check dam is shown in Figure 8-9.

Swales should be designed with minimum longitudinal slopes (1 to 2 percent) to promote infiltration and filtering characteristics but still maintain conveyance requirements to prevent flooding and local ponding in the swale. Check dams, as shown in Figures 8-8 and 8-9, are normally used when the longitudinal slope exceeds 2 to 4 percent. Figure 8-8 shows a perforated pipe enhancement to the swale that ensures the swale remains dry between storm events. Side slopes should be no greater than 2.5 to 1 but are optimally less than 4 to 1. A minimum bottom width of 0.75 m and minimum water depth of 0.5 m should be maintained. The maximum velocity in the swale should be 0.5 m/s. Where velocities are greater than 0.5 m/s the use of check dams (Figure 8-9) can promote infiltration and settling of pollutants. Grass should be local species or standard turf grass where a more manicured appearance is required. The grass should be allowed to grow higher than 75 mm so that suspended solids can be filtered effectively.

8.3.5 End-of-Pipe Stormwater BMPs

End-of-pipe stormwater BMPs provide water quality enhancement to stormwater prior to discharge into a receiving water body. A number of end-of-pipe alternatives are available for application depending on the characteristics of the upstream catchment and the requirements for water quality enhancement. Eight general categories of end-of-pipe BMP facilities are discussed:

- Wet ponds
- Dry ponds
- Wetlands
- Infiltration trenches
- Infiltration basins
- Filter strips
- Sand filters
- Oil/grit separators

All references to "wet ponds", "wetlands", or "dry ponds" assume that extended detention storage is provided. Extended detention refers to the dry or active storage provided by these facilities. Extended detention ponds reduce the rate of stormwater discharge by storing the stormwater runoff temporarily and releasing it at a controlled rate. Water quality treatment is provided through enhanced settling and biological processes. As such, extended detention storage provides benefits related to water quality, erosion protection, and flooding potential.

8.3.5.1 Wet Ponds

1. Purpose

The purpose of wet ponds is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potentials.

2. Description

Wet ponds can be created as an impoundment by either constructing an embankment or excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. The deep, permanent pond is the wet pond's primary water quality enhancement mechanism. Runoff entering the retention basin is designed to displace water already in the permanent pool and remain there until another storm event. Runoff entering the basin is slowed by the permanent pool and suspended pollutants are allowed to settle. Biologic processes, such as nutrient uptake by algae, are established in the permanent pool and help reduce concentrations of soluble contaminants. A vegetative planting strategy should provide shading, aesthetics, safety, and enhanced pollutant removal.

3. Applicability

A reliable source of runoff or groundwater discharge must be available to maintain the permanent pool of a wet pond. As such, wet ponds are generally considered for drainage areas greater than 5 ha. Because of a wet pond's ability to reduce soluble pollutants, it is generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Wet ponds may not be appropriate, or may require specialized design, where receiving water temperatures are a concern.

4. Effectiveness

Wet ponds are probably the most common end-of-pipe management facility for the control of peak runoff discharges and the enhancement of water quality. Wet ponds are very effective in controlling runoff and improving water quality when proper design considerations are made for those two objectives.

5. Water Quantity

As a detention facility, a wet pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Wet ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Wet ponds are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wet ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a pre-determined erosive threshold.

6. Water Quality

Wet ponds have been cited as providing the most reliable end-of-pipe BMP in terms of water quality treatment. This reliability is attributed to a number of factors including:

- Performance does not depend on soil characteristics
- Permanent pool prevents resuspension
- Permanent pool minimizes blockage of outlet
- Promotes biological removal of pollutants
- Permanent pool provides extended settling

Wet ponds have a moderate to high capacity to remove most urban pollutants depending on how large the volume of the permanent pool is in relation to the runoff produced from the contributing drainage area. The establishment of vegetative zones in and around a wet pond can enhance its pollutant removal capability.

7. Design considerations

Wet ponds must be designed to meet specific water quality and/or discharge rate objectives. Wet ponds designed to control peak discharge rates do not normally provide optimum water quality enhancement. Flood control or peak flow control wet ponds are typically designed to control the large infrequent event storms. Water quality wet ponds need to be designed to capture and treat the more frequent smaller storms with which the majority of the contaminant loadings are associated. Wet ponds can be designed to meet both flood control and water quality objectives.

One of the primary criteria for the proper design of a wet pond for peak runoff control is the provision of adequate detention storage volume. The primary design consideration for a wet pond for water quality enhancement is the settling velocity of the particulates in the stormwater entering the pond. The wet pond surface area is directly related to this required settling velocity. Ponds designed only for peak flow reduction do not normally provide adequate facility for water quality enhancement.

The design of a wet pond requires careful consideration of the required design objectives for flood control and water quality enhancement. Figure 8-10 illustrates some of the basic requirements for a wet pond. Detailed designed requirements should be evaluated for each individual application based on site specific constraints and objectives.

Some general design parameters are:

- Minimum water surface area of 2 ha
- Maximum sideslopes above active storage zone are 4:1 to 5:1
- Maximum interior sideslopes in active storage zone are 5:1 to 7:1
- Maximum exterior sideslopes are 3:1

Some water quality control design parameters are:

- Permanent pool sized to store the volume of runoff from a 25-mm storm over the contributing area

- Detention time of 24 hours

- Length to width ratio shall be from 4:1 to 5:1

- Minimum permanent pool depth of 2.0 m

- Maximum permanent pool depth of 3.0 m The maximum water level should be below adjacent house basement footings.

- Maximum active detention storage depth of 1.5 m

Some water quantity control design parameters are:

- 1-in-100-year storm stored within 2 m above the permanent pool (Alternatively, the 2 m can be used to store the 1-in-25-year storm. In such cases an emergency overflow drainage system should be constructed with the capacity to carry storm runoff from the 1-in-100-year storm event to receiving streams or downstream stormwater management facilities.)

- Detention time of 24 hours

Also, a wet ponds water quality control performance can be improved by providing a pretreatment sump or forebay and a backup water supply to maintain the minimum storage volume. During the design process, other design considerations should be evaluated that relate to ease of maintenance. The forebay should be designed with the following parameters:

- Length to width ratio of 2:1 or greater

- Forebay surface area not to exceed one-third of the permanent pool surface area
- Forebay length, L_{fb} as follows:

$$L_{fb} = [rQ_p/V_s]^{0.5}$$

where:

r	=	Length to width ratio of forebay
Q_p	=	Peak flow rate from the pond during the design quality storm (m^3/s)
V_s	=	Settling velocity (dependent on the desired particle size to settle)

- Dispersion length, L_{dis} as follows:

$$L_{dis} = (8Q)/(dV_f)$$

where:

Q	=	inlet flow rate (m^3/s)
d	=	depth of permanent pool in the forebay (m)
V_f	=	desired velocity at the end of the forebay

- Forebay Bottom Width, $W = L_{dis}/8$

- Forebay berm should be 0.15 to 0.3 metres below the permanent pool elevation

8.3.5.2 Dry Ponds

1. Purpose

The purpose of a dry pond is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potential.

2. Description

Dry ponds are impoundment areas constructed by an embankment or through excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. Unlike a wet pond, however, the lower stage is designed to empty completely between storm events.

3. Applicability

Dry ponds may be applied where topographical or planning constraints exist that limit the land available for wet ponds. Drainage areas greater than 5 ha are generally recommended for dry ponds. The use of dry ponds for combined water quantity and quality control is discouraged without the use of sediment forebays that include a permanent pool.

A dry pond's limited effectiveness in removing soluble contaminants is an important factor in considering its application. For example, in low-density residential areas where soluble nutrients from fertilizers and pesticides are a concern, dry ponds in isolation may not be appropriate.

4. Effectiveness

Dry ponds do not provide water quality enhancement because of the bottom scour that occurs with each storm event. Dry ponds do provide effective stormwater flow attenuation.

5. Water Quantity

As a detention facility, a dry pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Dry ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Dry ponds are relatively ineffective for volume reduction, although some evaporation may occur. Dry ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6. Water Quality

Because dry ponds have no permanent pool of water, the removal of stormwater contaminants in dry ponds is a function of the pond's drawdown time. The removal of soluble pollutants does not generally occur in a dry pond. Without a permanent pool, resuspension of contaminants is a concern. Dry ponds operating in a continuous mode are generally less effective at pollutant removal compared to wet ponds, whereas dry ponds operating in a batch mode have been reported to be similarly effective. In general, dry ponds should only be implemented if it is determined that a wet pond cannot be implemented due to topographical or planning constraints.

7. Design Considerations

The design of a dry pond has many site-specific requirements that must be considered. These design considerations are dependent on the constraints of a particular site and the objectives for the pond.

Figure 8-11 illustrates some of the basic requirements for a dry pond.

Some general design parameters are:

- Storage capacity for up to the 1-in-100-year storm
- Detention time of 24 hours
- Maximum active retention storage depth of 1.0 to 1.5 metres. The maximum water level should be below adjacent house basement footings.
- Maximum interior sideslopes of 4:1 to 5:1
- Maximum exterior sideslopes of 3:1
- Minimum freeboard of 0.6 m
- Minimum ratio of effective length to effective width of 4:1 to 5:1
- Minimum slope in the bottom of the pond of 1 percent (2 percent is preferred)

During the design process, other design considerations should be evaluated that relate to ease of maintenance and use. For example, a weeping tile system could be installed under the bottom of the pond to improve the rate at which the pond bottom dries out between storm events.

8.3.5.3 Constructed Wetlands

1. Purpose

By retaining runoff for a prolonged period of time and uptaking, altering, and storing pollutants, constructed wetlands serve to improve water quality and control peak discharge rates.

2. Description

There are five basic stormwater wetland designs: shallow marsh, pond/wetland, extended detention wetland, pocket wetland, and fringe wetland. All are essentially surface flow systems, with varying emergent marsh and deep pool habitat, hydraulic capacity, residence time, and travel routes.

Constructed wetlands can be created as an impoundment by either constructing an embankment or excavating a pit. Relatively deep permanent pools are maintained at the inlet and outlet and along low flow paths to minimize the resuspension and discharge of settled pollutants from the facility. Relatively shallow extended detention storage areas with extensive plantings (submergent and emergent) make up the majority of a constructed/artificial wetland's permanent storage. Sedimentation, filtration and biological processes account for the water quality benefits afforded by wetlands. Planting strategies are also implemented for shoreline fringe areas and/or floodfringe areas (if a combined facility) providing shading, aesthetics, safety, and enhanced pollutant removal.

3. Applicability

Generally wetlands' can be considered for drainage areas greater than 5 ha Because of a wetlands ability to reduce soluble pollutants, they are generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Constructed/artificial wetlands may not be appropriate, or may require specialized design, where receiving-water temperatures are a concern. The application of constructed/artificial wetlands may be further constrained by existing planning designations or topography that limits land availability. Potential ancillary benefits provided by wetlands include aviary, terrestrial, and aquatic habitat.

Wetland water treatment systems are not recommended for all applications. Such systems are most appropriate under the following conditions:

- Large tracts of suitable land are readily available.
- The influent does not contain high levels of industrial toxic pollutants as defined by provincial and federal agencies.
- There is a shortage of local groundwater or surface water supplies.
- A water body with impaired water quality is located in the area.
- The region has a history of wetland loss.
- Regulatory agencies are interested in the potential benefits of the technology.

4. Effectiveness

Stormwater wetland water treatment systems provide several major benefits:

- They require less maintenance and are less expensive to maintain than traditional treatment system.

- With proper design, portions of the wetland treatment system may provide additional wetland wildlife habitat, as well as recreational opportunities such as birdwatching, hiking, and picnicking.

- Wetland treatment systems are viewed as an asset by provincial and federal agencies in many regions and as a potentially effective method for replacing wetlands lost through agricultural practices, industrial and municipal development, and groundwater withdrawal.

5. Water Quantity

As a detention facility, a wetlands typically flatten and spread the inflow hygrograph, thus lowering peak discharges. Wetlands are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond/wetlands release rates for regional flood control. Wetlands are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wetlands are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6. Water Quality

In general, wetland water treatment systems have been found to lower BOD, TSS, and total nitrogen concentrations to 10 to 20 percent of the concentrations entering the systems. For total phosphorus, metals, and organic compounds, removal efficiencies vary widely, typically from 20 to 90 percent. Removal of these latter constituents appears to be limited by substrate type, the form of the constituents, the presence of oxygen, and the entire chemical makeup of the water to be treated.

7. Design Considerations

The design of a constructed wetland for dealing with urban stormwater requires a detailed study to determine from the outset what the goals of the wetland are. If the function is primarily to store water during storm events and release it later, then the size of the catchment area, permeability of the urban surfaces, and recorded flow rates will be used to determine the water volume storage capacity required. This, together with the expected frequency of large storm events, will provide an indication of the suggested drawdown rates for the wetland and the diameter of outflow pipes. If, on the other hand, improving water quality is a major goal, then subsurface water flow through one or more cells may be worth incorporating into the design specifications. Should the wetland operate in the fall, winter, and early spring as well as in summer? If so, then a configuration of wetland that is deep and permits water flow during low winter temperatures may be appropriate.

Several goals may be identified for a constructed wetland, but the available site may limit the achievement of all the goals. In this case priorities must be set. The general location of a constructed wetland is an important consideration. Is it to be constructed in a residential, industrial, or rural area? Considerations such as safety, aesthetics, potential toxic spills, or wildlife mean that different design criteria must be considered. To achieve water management goals, social as well as technical issues must be addressed, for "social" problems may be more difficult to solve than physical and technical ones, and managers should involve local interest groups in the early planning stages of projects.

It is important that a pretreatment area be provided for the collection of sediment and for the protection of the constructed wetland from accidental spills. Data is available on the construction of a pretreatment area for oil separation and sediment removal prior to allowing water to flow into a wetland.

A constructed wetland could contain a number of cells, either of similar construction and function, or of different structure and purpose. Figure 8-12 illustrates the major components of a constructed wetland.

General design considerations are:

- Wetland size should be approximately 5 percent of the watershed area that it will be servicing
- Approximately 10 percent of the wetland surface area should be a 1.5 to 2.0 m deep sediment forebay upstream of the wetland area for settleable solids removal
- Average permanent water wetland depth is 0.3 m with 1 m deep zones for flow redistribution and for fish and submerged or floating aquatic vegetation habitat
- Active storage is 0.3 to 0.6 m deep
- Vegetation can be cost effectively transplanted from local donor sites including ditches maintained by the Province and construction sites where small pocket wetlands are to be removed
- Length to width ratios can be as low as 1:1
- Shape of the treatment cell(s) can vary and depends on landscaping features required for attracting wildlife and for public enjoyment, and shape of available land
- Bottom slope of 0.5 to 1.0 percent is recommended and a flat bottom to promote sheet flow through the system
- Gravity flow is the preferred method of movement of water into, through, and out of the treatment wetland
- Incorporate a bypass that will collect first flush flows and divert high flows during extreme rainfall events around the wetland
- Regulated inflow and outflow structures are required that will take into account a wide range of rainfall intensities
- Landscaped features will provide an attractive park-like setting
- Ancillary benefits include provision for wildlife habitat, wildlife viewing opportunities, hiking areas, educational opportunities, and restoration of lost wetland areas
- Mosquito control includes introducing or making habitat available for baitfish (fathead minnows), dragon flies, purple martins, swallows, and bats
- Odour control is not required since the treatment wetlands, if designed properly, do not generate odours

- Nuisance wildlife including carp and muskrat will require control since they will destroy or consume the wetland vegetation and will, in the case of the carp, resuspend settled materials

- Freezing conditions during the winter months will not adversely affect the treatment wetland

- Design and implement with designated objectives constantly and clearly in mind.

- Design more for function than for form. A number of forms can probably meet the objectives, and the form to which the system evolves may not be the planned one.

- Design relative to the natural reference system(s), and do not over-engineer.

- Design with the landscape, not against it. Take advantage of natural topography, drainage patterns, etc.

- Design the wetland as an ecotone. Incorporate as much "edge" as possible, and design in conjunction with a buffer and the surrounding land and aquatic systems.

- Design to protect the wetland from any potential high flows and sediment loads.

- Design to avoid secondary environmental and community impacts.

- Plan on enough time for the system to develop before it must satisfy the objectives. Attempts to short-circuit ecological processes by over-management will probably fail.

- Design for self sustainability and to minimize maintenance.

8.3.5.4 Infiltration Trenches

1. Purpose

The purpose of an infiltration trench is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration. The three basic trench systems are complete exfiltration, partial exfiltration, and water quality exfiltration. Each system is defined by the volume of annual runoff diverted to the trench and the degree to which the runoff is exfiltrated into the soils. Infiltration trenches differ from on-lot infiltration systems in that they are generally constructed to manage stormwater flow from a number of lots in a developed area, not a single property.

2. Description

Infiltration trenches can be constructed at ground surface level to intercept overland flow directly, or constructed as a subsurface component of a storm sewer system. Infiltration trenches are generally composed of a clear stone storage layer and a sand or peat filter layer. There are other options for the type of filter used such as a non-woven filter fabric.

3. Application

Infiltration trenches are best utilized as recharge devices for compact residential developments (< 2 ha), rather than as a larger-scale, water quality treatment technique. Normally, infiltration trenches are not used in commercial or industrial areas because of the potential for high-contaminant loads or spills that may result in groundwater contamination.

4. Effectiveness

Infiltration trenches are effective in managing runoff from small residential areas. They are also effective when constructed under grassed swales to increase the infiltration potential of the swale. Clogging of the filter material can be a frequent problem if solids inputs are high and no pretreatment in the form of grassed filter strip for surface trenches or a suitable oil/grit separator for subsurface trenches is employed. Groundwater mounding may also become a problem if infiltration volumes are too high.

5. Water Quantity

Infiltration trenches provide marginal water quantity control. As such, the application of infiltration trenches is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

Infiltration trenches limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from infiltration trenches also provides recharge to the local groundwater that may in turn discharge to local streams, thus enhancing baseflows.

6. Water Quality

Pretreatment BMPs such as filter strips or oil/water separators are often used in combination with infiltration trenches to minimize the potential for suspended sediments to clog the trench. Infiltration trenches limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. Potential contamination of groundwater should be considered when examining runoff quality directed to the infiltration trench.

7. Design Considerations

A surface infiltration trench and a subsurface infiltration trench are shown in Figures 8-13 and 8-14, respectively. Infiltration trenches require groundwater levels and bedrock layers to be at least 1 m below the bottom of the infiltration trench. Soils must have a percolation rate of more that 15 mm/hr. A suitable filter fabric should be used to protect the stone storage media from clogging.

Careful consideration should be given to the volume of stormwater directed to the infiltration trench. Only sufficient volumes should be directed to the trench to allow, at a maximum, a forty eight hour drawdown period.

In a subsurface trench, a series of perforated pipes carries stormwater to the trench. A bypass pipe or flow path should be provided for flows in excess of the design capacity of the trench.

8.3.5.5 Infiltration Basins

1. Purpose

The purpose of an infiltration basin is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration.

2. Description

Infiltration basins are above-ground pond impoundment systems that promote recharge. Water percolating through an infiltration basin either recharges the groundwater system or is collected by an underground perforated pipe system and discharged at a downstream outlet. The appearance of an infiltration basin is similar to that of a wet or dry pond.

3. Applicability

Infiltration basins are generally considered for drainage areas less than 5 ha that have permeable soils. As with wet or dry ponds, an infiltration basin can be designed as a multi-stage facility to achieve various stormwater management objectives. Infiltration basins should be used in residential areas only. Runoff from industrial or commercial land areas is generally of poor quality and could contaminate groundwater.

4. Effectiveness

Infiltration basins have a very high rate of failure. Most failures can be attributed to poor site selection, poor design, poor construction techniques, large drainage area, and lack of maintenance. One of the main problems inherent in infiltration basins is that large volumes of water from a large catchment area are expected to infiltrate over a very small surface area. This leads to numerous problems and general failure of these basins.

5. Water Quantity

Infiltration basins are generally ineffective for water quantity control. They only infiltrate limited volumes of water from generally large catchment areas and must be provided with an overflow structure to discharge excess flow. As such, the application of infiltration basins is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

6. Water Quality

The application of pretreatment to reduce sediment loadings and a bypass to restrict flows during certain periods (road sanding/salting, local excavation works, facility maintenance) is recommended to improve long-term infiltration basin performance.

7. Design Considerations

A typical infiltration basin is illustrated in Figure 8-15. Infiltration basin design considerations must include provision for construction at the end of the development construction. Also, compaction of the basin and smearing of the basin native material must be avoided. The basin must be constructed with a maximum water storage depth of 0.6 m to avoid compaction, and the groundwater table should be a minimum of 1.0 m below the infiltration layer. Any area bedrock should also be a minimum of 1.0 m below the infiltration layer. Planting in the basin should include grasses and legumes to maintain or enhance the pore spaces in the soil.

8.3.5.6 Filter Strips

1. Purpose

Filter strips are engineered conveyance systems that are designed to remove pollutants from overland runoff. By reducing overland flow velocities, the time of concentration and infiltration are increased, thereby slightly reducing the volume of runoff and minimally controlling discharge rates.

2. Description

There are two general types of filter strips: grass and forested. Both consist of a level spreader, which ensures level flows, and abundant vegetative plantings. The vegetative plantings promote pollutant filtration and infiltration of stormwater. Filter strips are generally best implemented adjacent to a buffer strip, watercourse, or drainage swale, as discharge from a filter strips will be a sheet flow and thus difficult to convey in a traditional stormwater conveyance system.

3. Applicability

Filter strips are best applied as one of a combination of BMPs as the maintenance of sheet flow through the vegetation, and thus consistent water quality benefits, has been difficult to maintain in practice.

4. Effectiveness

Limited filter strip performance data are available in the literature although it is generally thought that properly designed filter strips are capable of removing a high percentage of stormwater particulates.

5. Water Quantity

Filter strips may slightly reduce the volume of runoff by inducing infiltration.

6. Water Quality

Although filter strips have been shown to be somewhat effective in removing sediment and pollutant loads in urban stormwater runoff, the ability to maintain sheet flow through the vegetation over the long term has been questioned.

7. Design Considerations

A schematic of a grassed and wooded filter strip is shown in Figure 8-16. The filter strip requires a level spreader with available upstream storage to regulate the discharge rate and depth of flow through the filter strip. The ideal slope for a filter strip is less than 5.0 percent over a distance of 10 to 20 m in the direction of flow.

8.3.5.7 Sand Filters

1. Purpose

Sand filters are above or below ground end-of-pipe treatment devices that promote pollutant removal from overland runoff or storm sewer systems. Sand filters do not provide a recharge benefit as filtered stormwater is discharged to the storm sewer or receiving water.

2. Description

Sand filters can be constructed either above or below ground as an end-of-pipe BMP. They are most commonly constructed with impermeable liners to guard against native material clogging pore spaces and to prevent filtered water from entering the groundwater system. Water that infiltrates through the filer is collected by a previous pipe system and conveyed to a downstream outlet. Some designs incorporate a layer of peat to enhance pollutant removal capabilities of the sand filter, thus making discharge to an infiltration trench a possibility.

3. Applicability

Sand filters can be constructed either above or below ground as an end-of-pipe BMP and are generally only appropriate for relatively small drainage areas (< 5 ha). Also, very little is known in regard to sand filter performance and cold-climate operation and maintenance.

4. Effectiveness

This method of water quality enhancement should not be generally applied without a detailed feasibility assessment.

5. Water Quantity

Sand filters are not suitable for water quantity control as they should not be designed to handle large influent flows.

6. Water Quality

Sand filters have been found to be effective in removing pollutants, however, little is know about their performance in winter or freshet conditions.

7. Design Considerations

A sand filter application is illustrated in Figure 8-17. Sand filters can be constructed as surface filters or subsurface filters as part of the stormwater conveyance system. Surface filters are normally covered by a grass layer. Filters are lined with impermeable membranes to restrict clogging of the filter material by native material.

8.3.5.8 Oil/Grit Separators

1. Purpose

Oil/grit separators are a variation of traditional settling tanks. They are designed to capture sediment and trap hydrocarbons suspended in runoff from impervious surfaces as the runoff is conveyed through a storm sewer network.

2. Description

An Oil/grit separator is a below ground, pre-cast concrete structure that takes the place of a conventional manhole in a storm drain system. The separator implements the use of permanent pool storage in the removal of hydrocarbons and sediment from stormwater runoff before discharging into receiving waters or storm sewer systems.

3. Applicability

Oil/grit separators are typically applied to urban based drainage areas (<5ha) where ponds or wetlands are not feasible or cost effective. Separators are best applied in areas of high impervious cover where there is a potential for hydrocarbon spills and polluted sediment discharges. Typical applications include parking lots, commercial & industrial sites, petroleum service stations, airports, and residential developments (pre-treatment of ponds/wetlands or as part of a treatment train).

4. Effectiveness

Oil/grit separators can be effective for treatment of stormwater pollution at its source. Source control is favorable for water quality control since the dilution of pollutants in stormwater becomes problematic in terms of effective treatment as the drainage area increases. Depending on land use, drainage area, site conditions, and hydrology, some oil/grit separators may be effective in reducing TSS. See Table 8-2 for oil/grit separator design types and characteristics.

5. Water Quantity

Oil/grit separators implement the use of permanent pool storage for removal of stormwater pollution. However, they are not designed to provide extended detention storage, and thus provide little flow attenuation.

6. Water Quality

Oil/grit separators vary in design and performance. Separators that do not incorporate a high flow bypass have been found to be generally ineffective in removing/containing hydrocarbon and sediment pollutants, because of a continuous process of resuspension and settling of solids.

7. Design Considerations

Three chambered oil/grit separators operate most effectively when constructed offline. A flow splitter should be used to direct excess flow back to the conveyance system or to some other control practice. Only low flows should be directed to the separator.

Bypass separators are installed online, and high flows do not affect the performance of the unit.

See Figures 8-18 and 8-19 for illustrations of the two types of oil/grit separators.

8.3.6 BMP Screening and Selection

8.3.6.1 Initial Screening

There are a range of stormwater BMP options available for most applications. The selection of an appropriate BMP or group of BMPs depends first on the objectives for stormwater management defined for a particular catchment area, as well as the constraints placed on the feasibility of particular BMPs by physical site factors.

Once the objectives for stormwater management are well defined and the site constraints are understood individual BMPs can be evaluated in terms of their overall effectiveness as a stormwater control facilities. The evaluation of overall effectiveness must include both water quantity and water quality objectives.

Also, each stormwater management BMP has associated with it certain advantages and disadvantages that may allow the viable options for stormwater management to be reduced for a particular development area.

Table 8-2 summarizes the advantages and disadvantages of a number of BMPs.

8.3.6.2 Physical Constraints

Site characteristics may be the factor that will ultimately determine the applicability of individual or combinations of BMPs. Physical factors that need to be assessed in evaluating the suitability of BMPs include:

- Topography
- Soils stratification
- Depth to bedrock
- Depth to seasonably high water table
- Drainage area

Table 8-3 summarizes physical constraints associated with various BMP types.

8.3.6.3 Final Screening

In the initial screening phase the options for BMPs were limited by particular disadvantages and site constraints. The list of BMP options that are still considered feasible are further screened by the application of specific objectives that must be met as part of the development including:

- Water quality
- Flooding
- Erosion
- Recharge

The performance of the BMPs in regard to the objectives for stormwater management are shown in Table 8-4.

8.3.6.4 Water Quality Control and Enhancement Opportunities

In many areas of development, stormwater management practices must meet stringent water quality objectives to protect sensitive receiving waters. Water quality objectives can be defined for a stormwater management system and then appropriate BMPs can be selected from the prescreened list that will meet the water quality objectives.

The reported effectiveness, to remove pollutants, of a number of BMPs are shown in Tables 8-5.

TABLE 8-2

BMP ADVANTAGES AND DISADVANTAGES

BMP	Advantages	Disadvantages
Wet pond	• Capable of removing soluble as well as solid pollutants • Provides erosion control • Habitat, aesthetic, and recreation opportunities provided • Relatively less frequent maintenance schedule	• More costly than dry ponds • Permanent pool storage requires larger land area • Could have negative downstream temperature impacts • Could be constrained by topography or land designations • Sediment removal relatively costly when required
Dry pond	• Batch mode has comparable effectiveness to wet ponds • Not constrained by land area required by wet ponds • Can provide recreational benefits	• Potential resuspension of contaminants • More expensive O&M costs than wet ponds (batch mode)
Wetlands	• Pollutant-removal capability similar to wet ponds • Offers enhanced nutrient-removal capability • Potential ancillary benefits, including aviary, terrestrial, and aquatic habitat	• Requires more land area than wet ponds • Could have negative downstream temperature impacts • Could be constrained by topography or land designations • Potential for some nuisance problems
Infiltration trenches	• Potentially effective in promoting recharge and maintaining low flows in small areas • May be appropriate as secondary facility where maintenance of groundwater recharge is a concern • No thermal impact • No public safety concern	• Appropriate only to small drainage areas (<2 ha) and residential land uses • Constrained by native soil permeabilities • Usually requires pretreatment device • Potential contamination of groundwater must be investigated • Generally ineffective for water quantity control • High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance
Infiltration basins	• Potentially effective in promoting recharge and maintaining low flows in small areas • May be appropriate as secondary facility where maintenance of groundwater recharge is a concern • No thermal impact • No public safety concern	• Appropriate only to relatively small drainage areas (<5 ha) and residential land uses • Constrained by native soil permeabilities • Pretreatment is recommended • Potential contamination of groundwater must be investigated • Generally ineffective for water quantity control • High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance

TABLE 8-2

BMP ADVANTAGES AND DISADVANTAGES

BMP	Advantages	Disadvantages
Filter strips	• Water quality benefits may be realized if part of overall SUM plan (i.e., as secondary facility) • Effective in filtering out suspended solids and intercepting precipitation • May reduce runoff by reducing overland flow velocities, increasing time of concentration, and increasing infiltration • Can create wildlife habitat • No thermal impact	• Limited to small drainage areas (<2 ha) with little topographic relief • Uniform sheet flow through vegetation difficult to maintain • Effectiveness in freeze/thaw conditions questionable
Sand filters	• Generally effective in removing pollutants, are resistant to clogging and are easier/less expensive to retrofit compared to infiltration trenches	• Not suitable for water quantity control • Generally applicable to only small drainage areas (<5 ha) • Do not generally recharge groundwater system • May cause aesthetic/odour problems • O&M costs generally higher than other end-of-pipe facilities
Oil/Grit Separators (3-Chamber Separator)	• Offline, 3-chamber (oil, grit, discharge) separators may be appropriate for commercial, industrial, large parking, or transportation-related areas less than 2 ha	• Scour and resuspension of trapped pollutants in heavy rainfall events • Difficult to maintain • Relatively high O&M costs • Online design of 3-chamber separators has resulted in poor pollutant removal performance
Oil/Grit Separators (Bypass Separator)	• Bypass prevents the scouring and resuspension of trapped pollutants in heavy rainfall events • Effective in removing sediment load when properly applied as a source control for small areas • Effective in trapping oil/grease from run off	• Relatively high capital costs compared to manholes • Applicable for drainage areas less than 5 ha

TABLE 8-3

PHYSICAL BMP CONSTRAINTS

BMP	Criteria				
	Topography	Soils	Bedrock	Groundwater	Area
On-Lot BMP					
Flat lot grading	<5%	none	none	none	none
Soak-away pit	none	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Rear yard infiltration	<2%	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Conveyance BMP					
Grassed swales	<5%	none	none	none	none
Perforated pipes	none	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	none
Pervious catchbasins	none	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	none
End-of-Pipe BMP					
Wet pond	none	none	none	none	>5 ha
Dry pond	none	none	none	none	>5 ha
Wetland	none	none	none	none	>5 ha
Infiltration basin	none	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	<5 ha
Infiltration trench	none	loam (min. infiltration rate >15 mm/h)	>1 m below bottom	>1 m below bottom	<2 ha
Filter strips	<10%	none	none	>0.5 m below bottom	<2 ha
Sand filters	none	none	none	>0.5 m below bottom	<5 ha
Oil/grit separators	none	none	none	none	<1 ha

TABLE 8-4

POTENTIAL BMP OPPORTUNITIES

Stormwater BMP	Water Quality	Flooding	Erosion	Recharge
Lot Level BMPs				
Lot grading	♦	♦	♦	!
Roof leader ponding	♦	♦	♦	!
Roof leader soak-away pits	♦	♦	♦	!
Conveyance BMPs				
Pervious pipes	! *	♦	♦	!
Pervious catchbasins	! *	♦	♦	!
Grassed swales	!	♦	!	♦
End-of-Pipe BMPs				
Wet pond	!	!	!	"
Dry pond	♦	"	!	"
Dry pond with forebay	!	!	!	"
Wetland	!	!	!	"
Sand filter	!	♦	♦	"
Infiltration trench	♦*	♦	♦	!
Infiltration basin	♦*	♦	♦	!
Vegetated filter strip	!	"	♦	♦
Buffer strip	♦	"	♦	♦
Special purpose BMP				
Oil/grit separator	♦	"	"	"

! Highly effective (primary control)
♦ Limited effectiveness (secondary control)
" Not effective
* May have adverse effects
From MOEE, 1994

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Infiltration Basin	Average:	75	65	80	65	65	65	Soil percolation rates Basin surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodword-Clyde, 1966
	Reported Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-80	50-80	50-80	50-80	50-80	50-80		
	No. of Values Considered:	7	7	7	4	4	4		
Infiltration Trench	Average:	75	60	55	65	65	65	Soil Percolation rates Trench surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodword-Clyde, 1966; Kuo et al 1968; Lugbill, 1990
	Reported Range:	45-100	40-100	(110)-100	45-100	45-100	45-100		
	Probable Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-90	50-90	50-90	50-90	50-90	50-90		
	No. of Values Considered:	9	9	9	4	4	4		

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Vegetated Filter Strip	Average:	65	40	40	40	45	60	Runoff volume Slope Soil infiltration rates Vegetative cover Buffer length	IEP, 1991 Casman, 1990 Glick et al, 1991 VADC, 1987 Minnesota PCA, 1989 Scheuler, 1967 Hartigan et al 1969
	Reported Range:	20-80	0-95	0-70	0-60	20-90	30-90		
	Probable Range:	40-90	30-80	20-60	-	30-80	20-50		
	No. of Values Considered:	7	4	3	2	3	3		
Grass Swale	Average:	60	20	10	25	70	60	Runoff volume Slope Soil infiltration rates Vegetative cover Swale length Swale geometry	Yousel et al, 1965 Dupuls, 1985 Washington State, 1968 Schuerer, 1967 British Columbia Res. Corp, 1991 EPA, 1983 Whelen et al, 1988 PIN, 1966 Caeman, 1990
	Reported Range:	0-100	0-100	0-40	25	3-100	50-80		
	Probable Range:	20-40	20-40	10-30	-	10-20	10-20		
	No. of Values Considered	10	8	4	1	10	7		
Porous Pavement	Average:	35	5	20	5	15	5	Maintenance Sedimentation storage volume	Pitt, 1965 Field, 1985 Schueler, 1967
	Reported Range:	0-95	5-10	5-55	5-10	10-25	5-10		
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Concrete Grid Pavement	Average:	90	90	90	90	90	90	Percolation rates	Day, 1961 Smith et al, 1961 Schueler, 1967
	Reported Range:	65-100	65-100	65-100	65-100	65-100	65-100		
	Probable Range:	60-90	60-90	60-90	60-90	60-90	60-90		
	No. of Values Considered:	2	2	2	2	2	2		
Sand Filter/Filtrat-ion Basin	Average:	80	50	35	55	60	65	Treatment volume Filtration media	City of Austin, 1986 Environmental and Conservation Service Department, 1990
	Reported Range:	60-95	0-90	20-40	45-70	30-90	50-80		
	Probable Range:	60-90	0-80	20-40	40-70	40-80	40-80		
	No. of Values Considered:	10	6	7	3	5	5		
Water Quality Inlet	Average:	35	5	20	5	15	5	Maintenance Sedimentation storage volume	Pitt, 1965 Field, 1965 Schueler, 1967
	Reported Values:	0-95	5-10	5-55	5-10	10-25	5-10		
	Probable Values:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Water Quality Inlet with Sand Filter	Average:	80	NA	35	55	80	65	Sedimentation storage volume Depth of media	Shaver, 1991
	Reported Range:	75-85	NA	30-45	45-70	70-90	50-80		
	Probable Range:	70-90	_	30-40	40-70	70-90	50-80		
	No. of Values Considered:	1	0	1	1	1	1		
Oil/Grit Separator	Average:	15	5	5	5	15	5	Sedimentation storage volume Outlet configurations	Pitt, 1965 Schueler, 1967
	Reported Range:	0-25	5-10	5-10	5-10	10-25	5-10		
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	2	1	1	1	1	1		
Extended-Detention Dry Pond	Average:	45	25	30	20	50	20	Storage volume Detention time Pond shape	MWCOG, 1983 City of Austin, 1990 Schueler and Heinrich, 1965 Pope and Hess, 1989 OWML, 1967 Wollnold and Stack, 1990
	Reported Range:	5-90	10-55	20-60	0-40	25-65	(-40)-65		
	Probable Range:	70-90	10-60	20-60	30-40	20-60	40-60		
	No. of Values Considered:	6	6	4	5	4	5		

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Wet Pond	Average:	60	45	35	40	75	80	Pond volume Pond shape	Wotzka and Oberta, 1966 Yousel et al, 1968 Cullum, 1985 Driscoll, 1983 Driscoll, 1986 MWCOG, 1963 OWML, 1963 Yu and Benemouflok, 1986 Hother, 1989 Martin, 1966 Dowman et al, 1969 OWML, 1962 City of Austin, 1990
	Reported Range:	(-30)-91	10-85	5-85	5-90	10-85	10-95		
	Probable Range:	50-90	20-90	10-90	10-90	10-95	20-95		
	No. of Values Considered:	18	18	9	7	13	13		
Extended-Detention Wet Pond	Average:	80	65	55	NA	40	20	Pond volume Pond shape Detention time	Ontario Ministry of the Environment, 1991 cited in Schueler et al 1992
	Reported Range:	50-100	50-60	55	NA	40	20		
	Probable Range:	50-95	50-90	10-90	10-90	10-95	20-95		
	No. of Values Considered:	3	3	1	0	1	1		

TABLE 8-5

EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Constructed Stormwater Wetlands	Average:	65	25	20	50	65	35	Storage volume Detention time Pool shape Wetlands biota Seasonal variation	Harper et al, 1966 Brown, 1985 Wotzka and Oberta, 1966 Hickock et al, 1977 Burten, 1967 Martin, 1966 Morris et al, 1961 Sherberger and Davis, 1962 ABAG, 1979 Oberts et al, 1969 Rushton and Dye, 1990 Hay and Barrett, 1991 Martin and Smool, 1986 Ralnelt et al, 1990 cited in Woodward and Clyde, 1991
	Reported Range:	(-20)-100	(-120)-100	(-15)-40	20-80	30-95	(-30)-60		
	Probable Range:	50-90	(-5)-80	0-40	-	30-95	-		
	No. of Values Considered:	23	24	6	2	10	8		

NA Not available
[a] Design criteria: storage volume equals 80% avg. runoff volume, which completely drains in 72 hours; maximum depth = 6 ft.; minimum depth = 2 ft.
[b] Design criteria: storage volume equals 90% avg. runoff volume, which completely drains in 72 hours; maximum depth = 5 ft.; minimum depth = 3 ft.; storage volume = 40% excavated trench volume
[c] Design criteria: flow depth < 0.3 ft.; travel time > 5 min.
[d] Design criteria: Low slope and adequate length
[e] Design criteria: minimum extended detention time 12 hours
[f] Design criteria: minimum area of wetland equal 1% of drainage area
[g] No information was available on the effectiveness of removing oil and grease
[h] Also reported as 90% TSS removed
[i] Also reported as 50% TSS removed

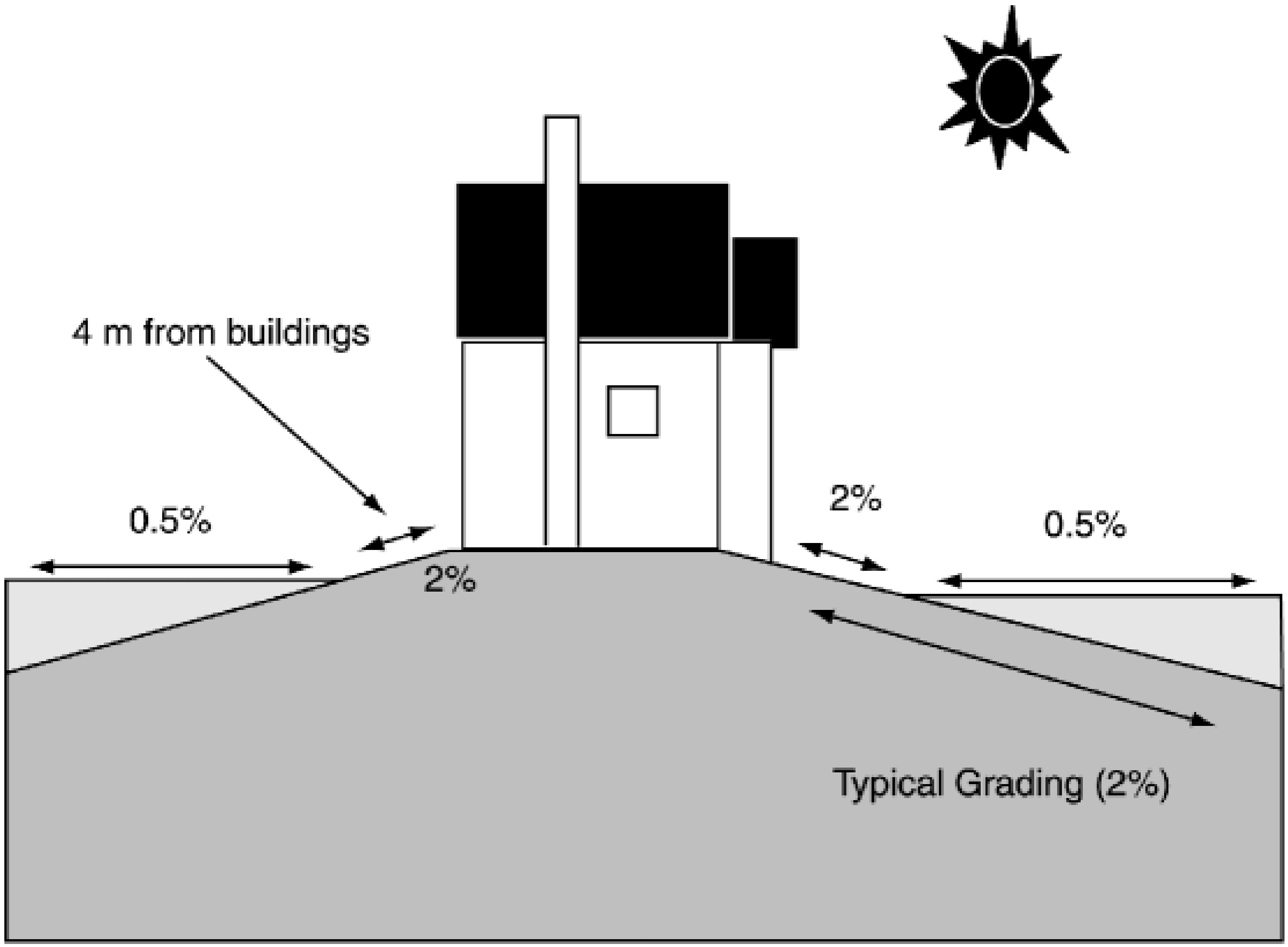


Figure 8.1
Lot Grading Guidelines

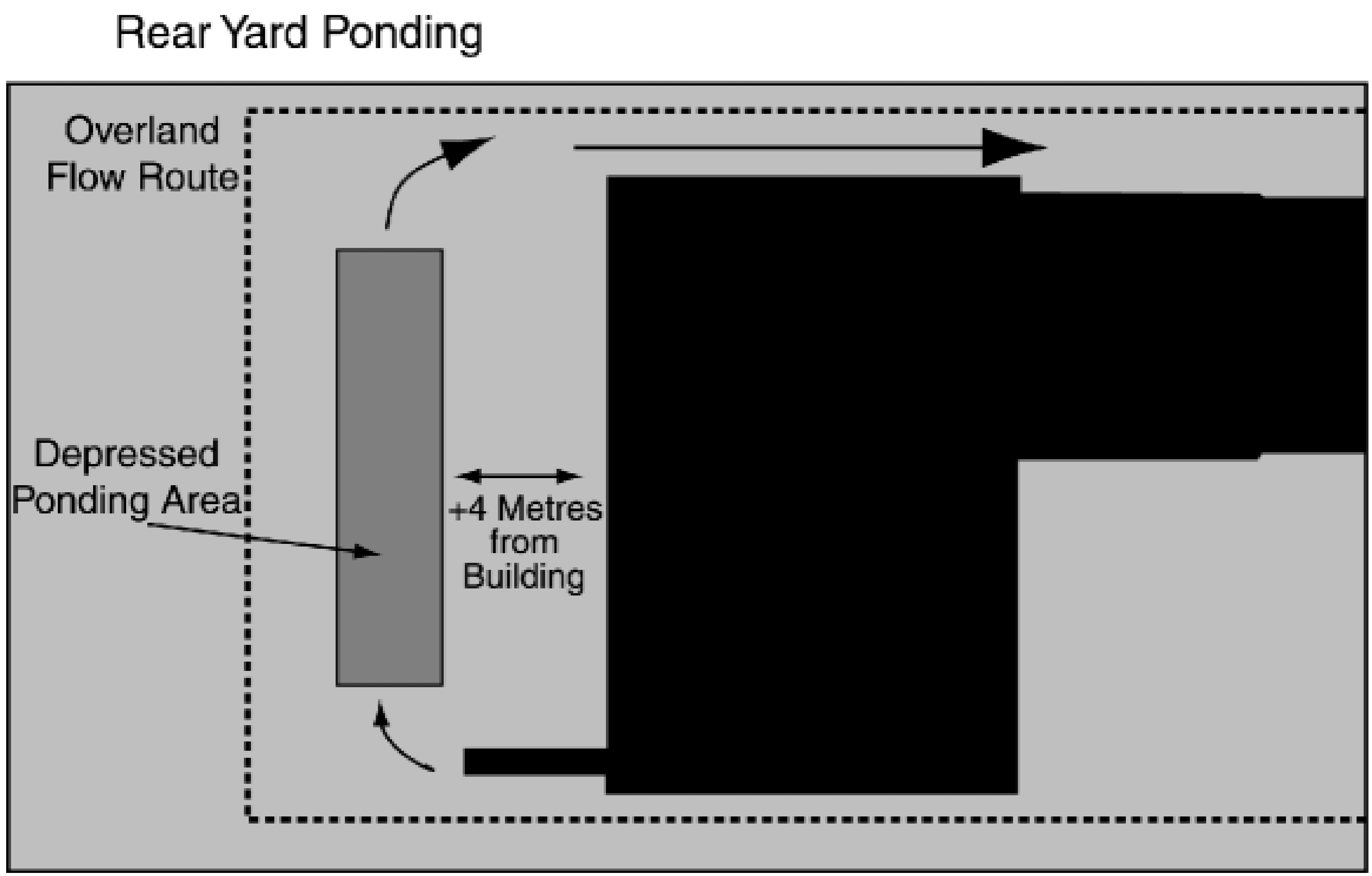


Figure 8.2
Rear Lot Ponding Guidelines

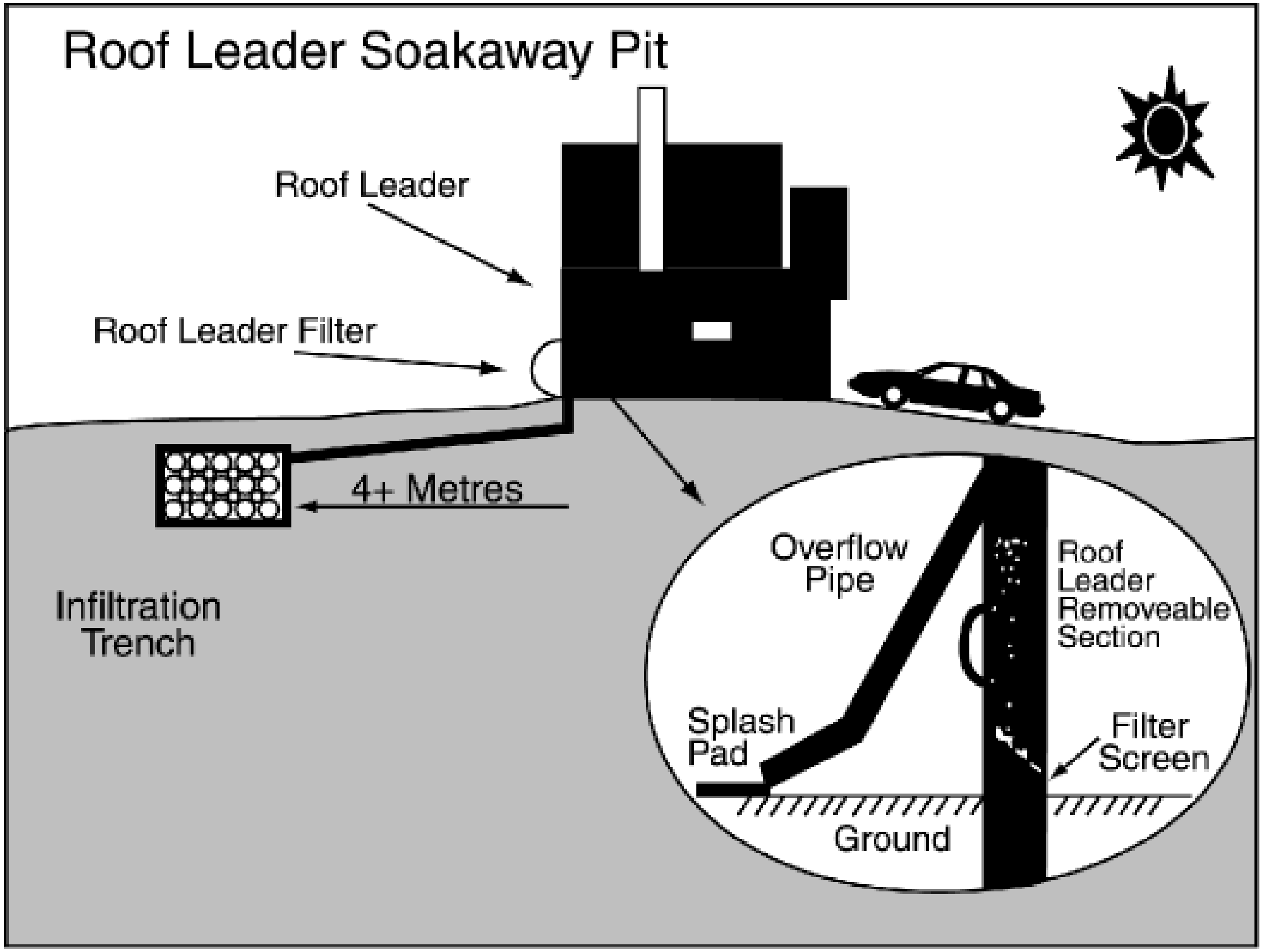


Figure 8.3
Infiltration System with Roof Leader Filter

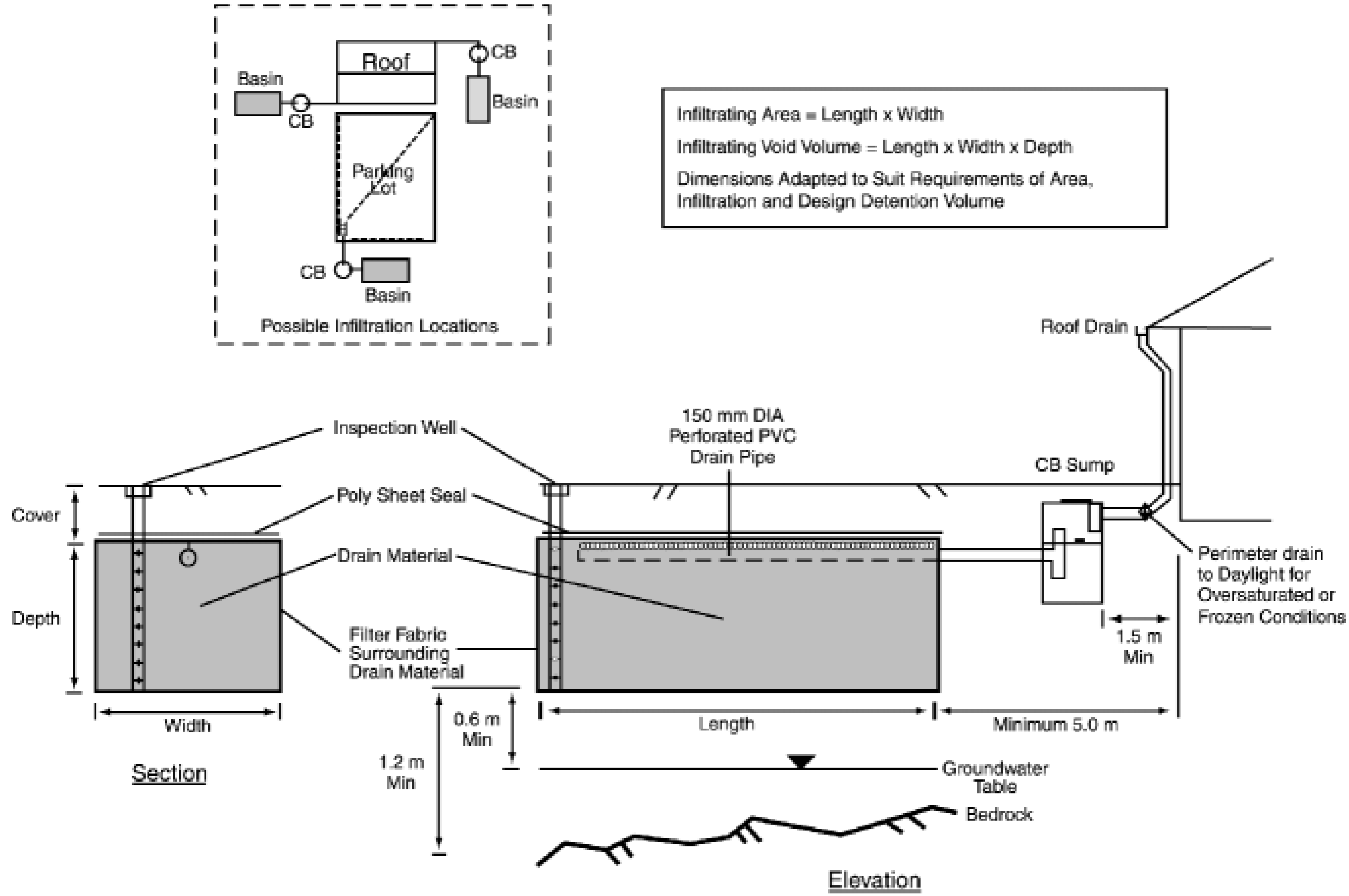


Figure 8.4
Infiltration System with Pretreatment Sump

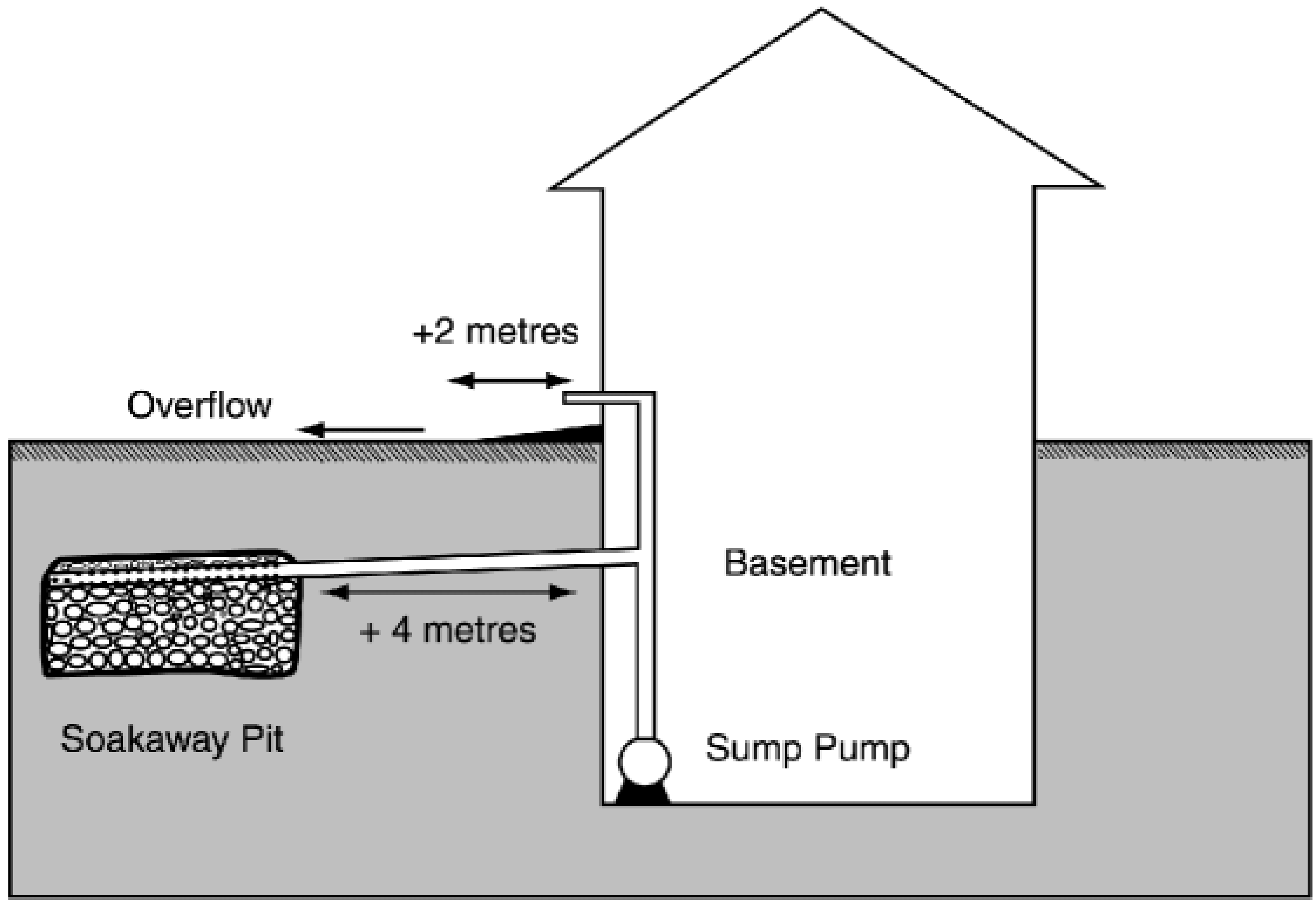


Figure 8.5
Sump Pump Foundation Drainage

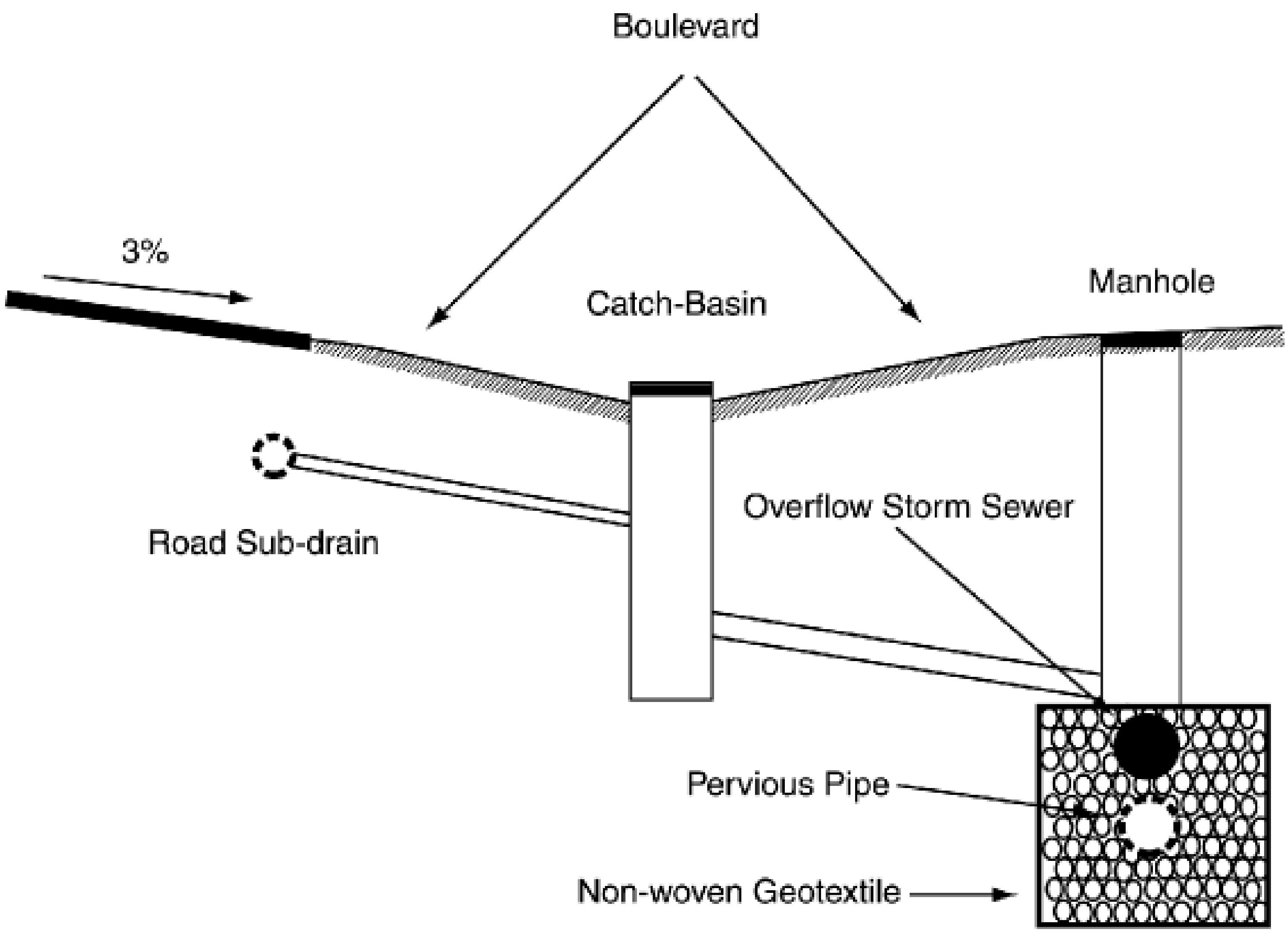


Figure 8.6
Pervious Pipe System

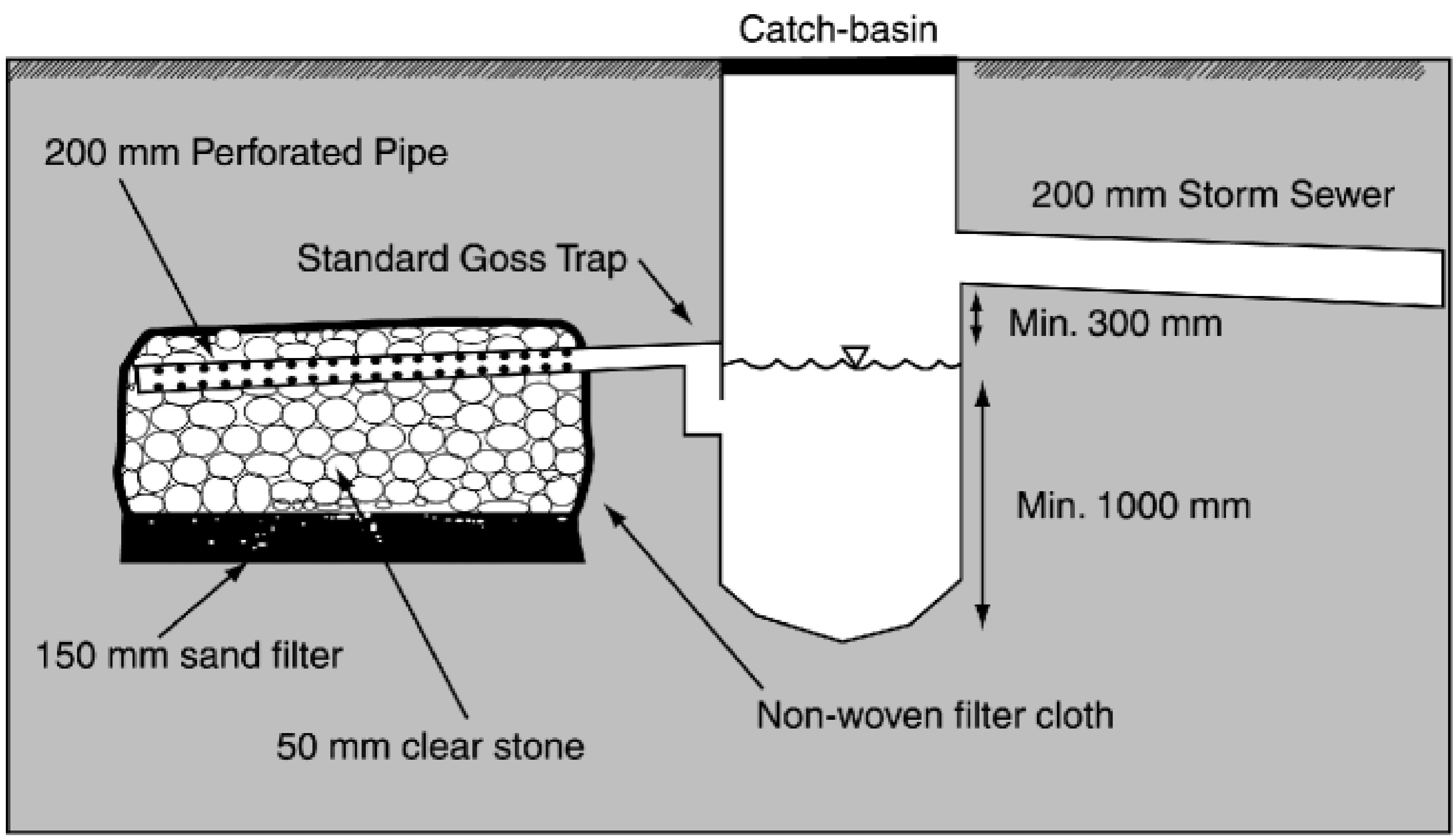


Figure 8.7
Pervious Catch-Basin

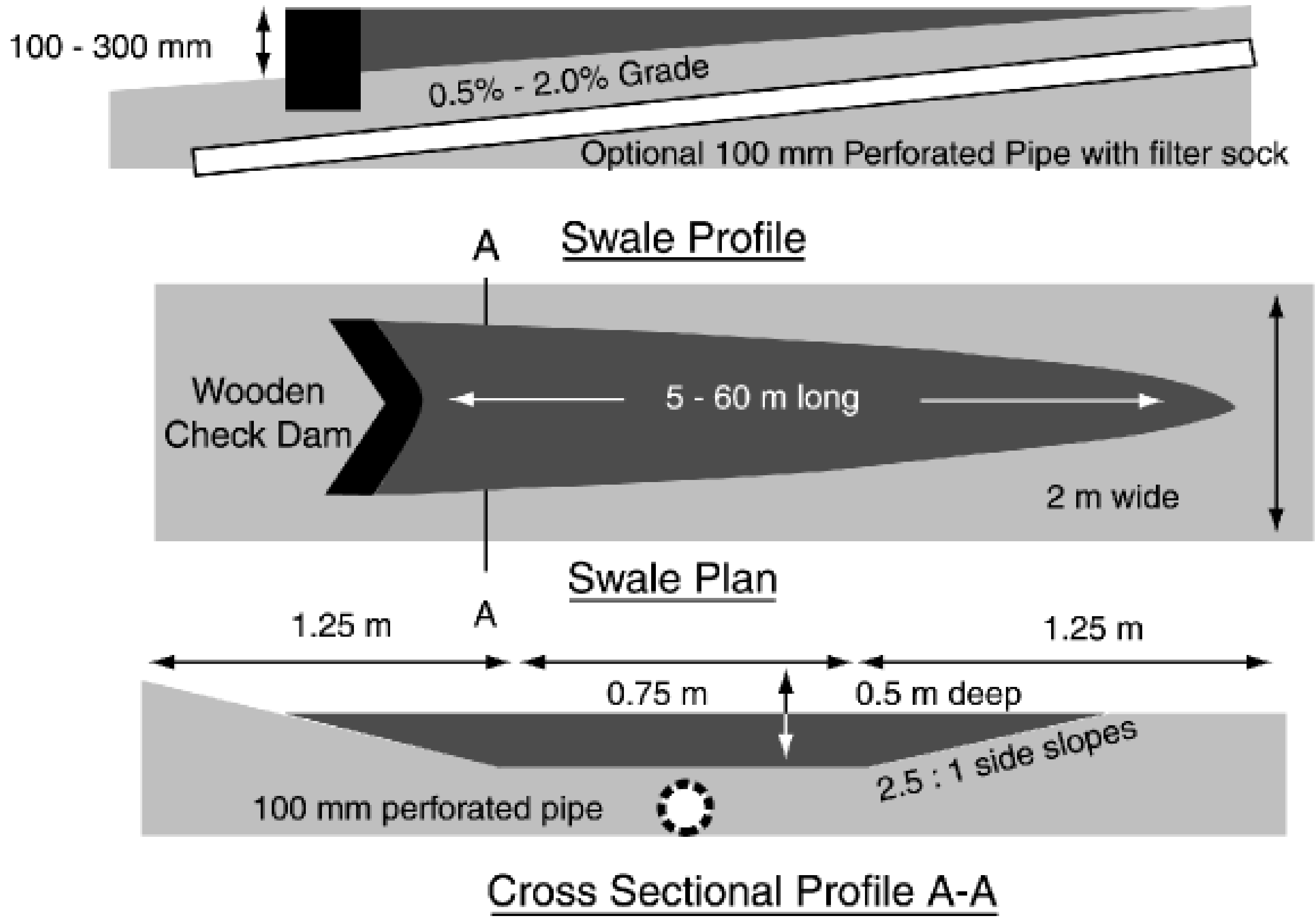


Figure 8.8
Grass Swale Design

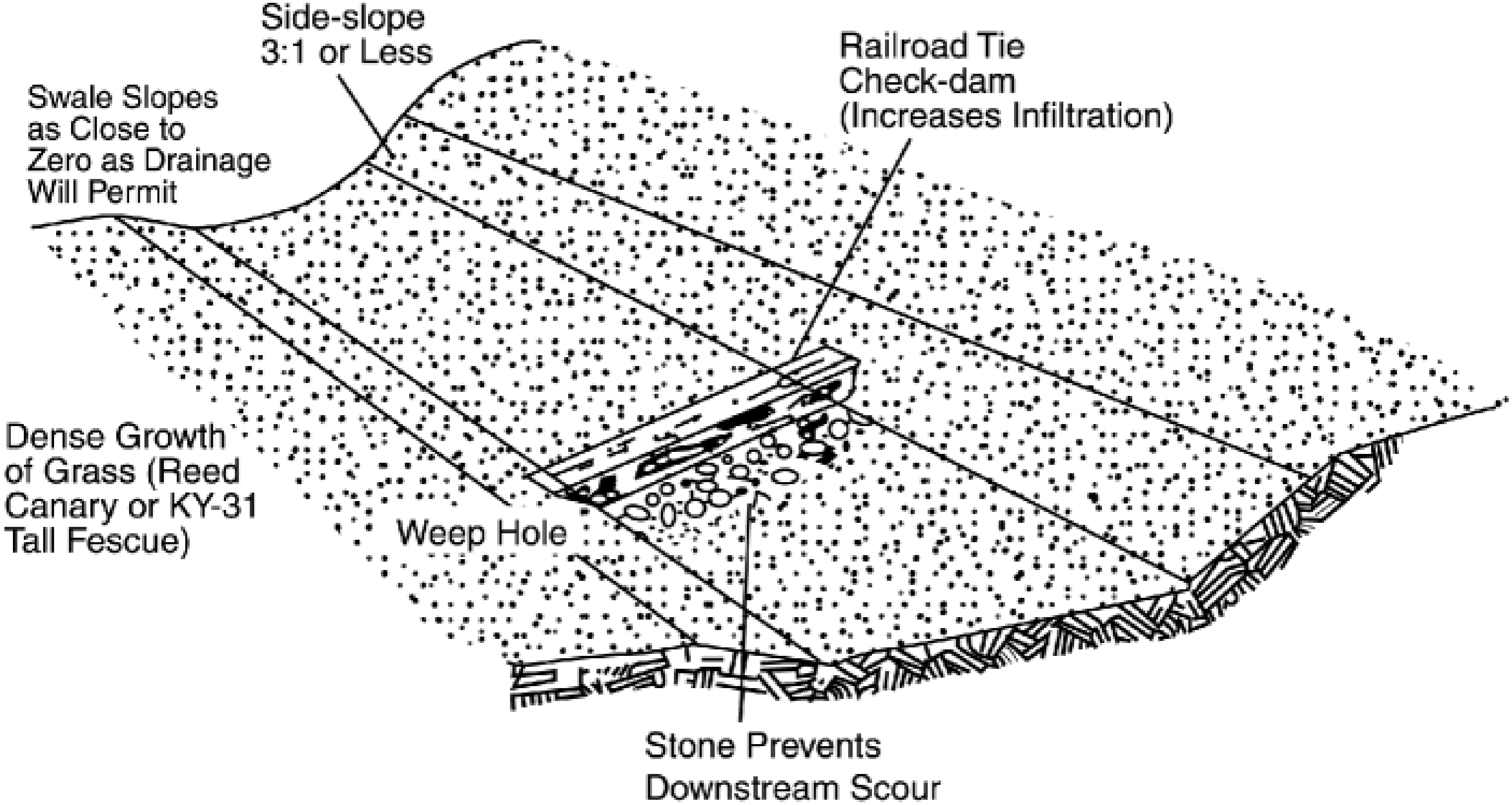


Figure 8.9
Grass Swale with Check Dam

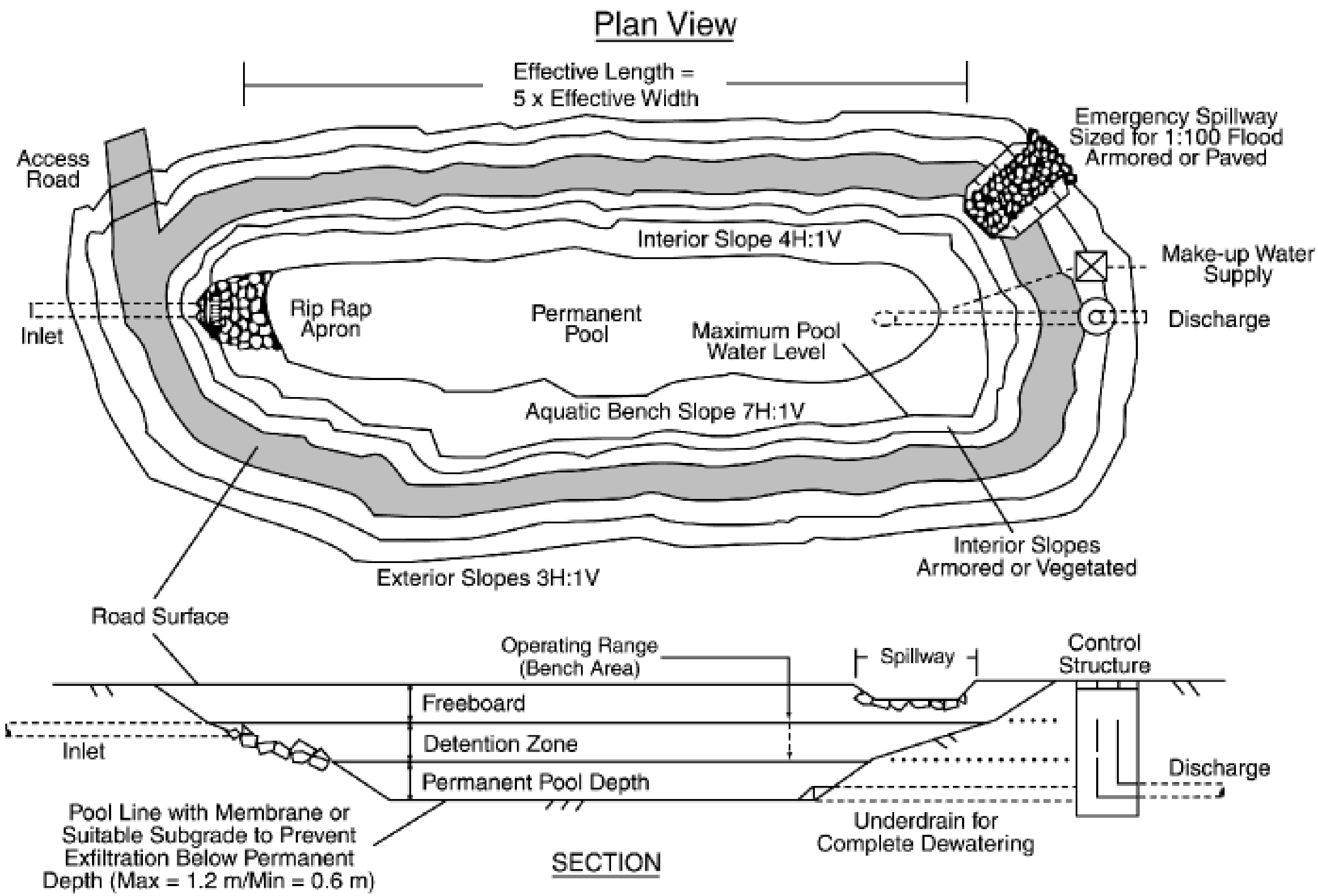


Figure 8.10
Wet Detention Pond Plan and Sections

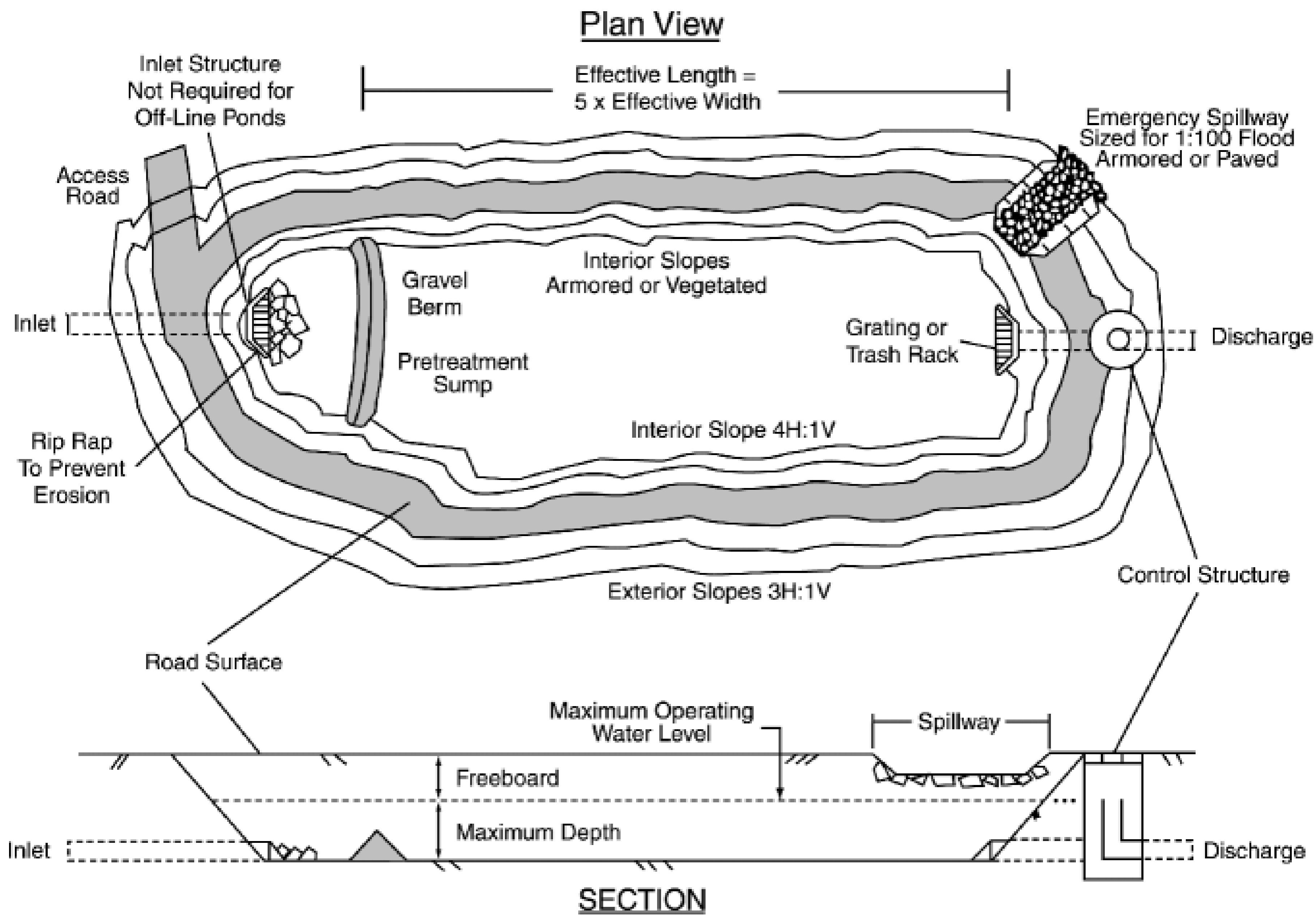
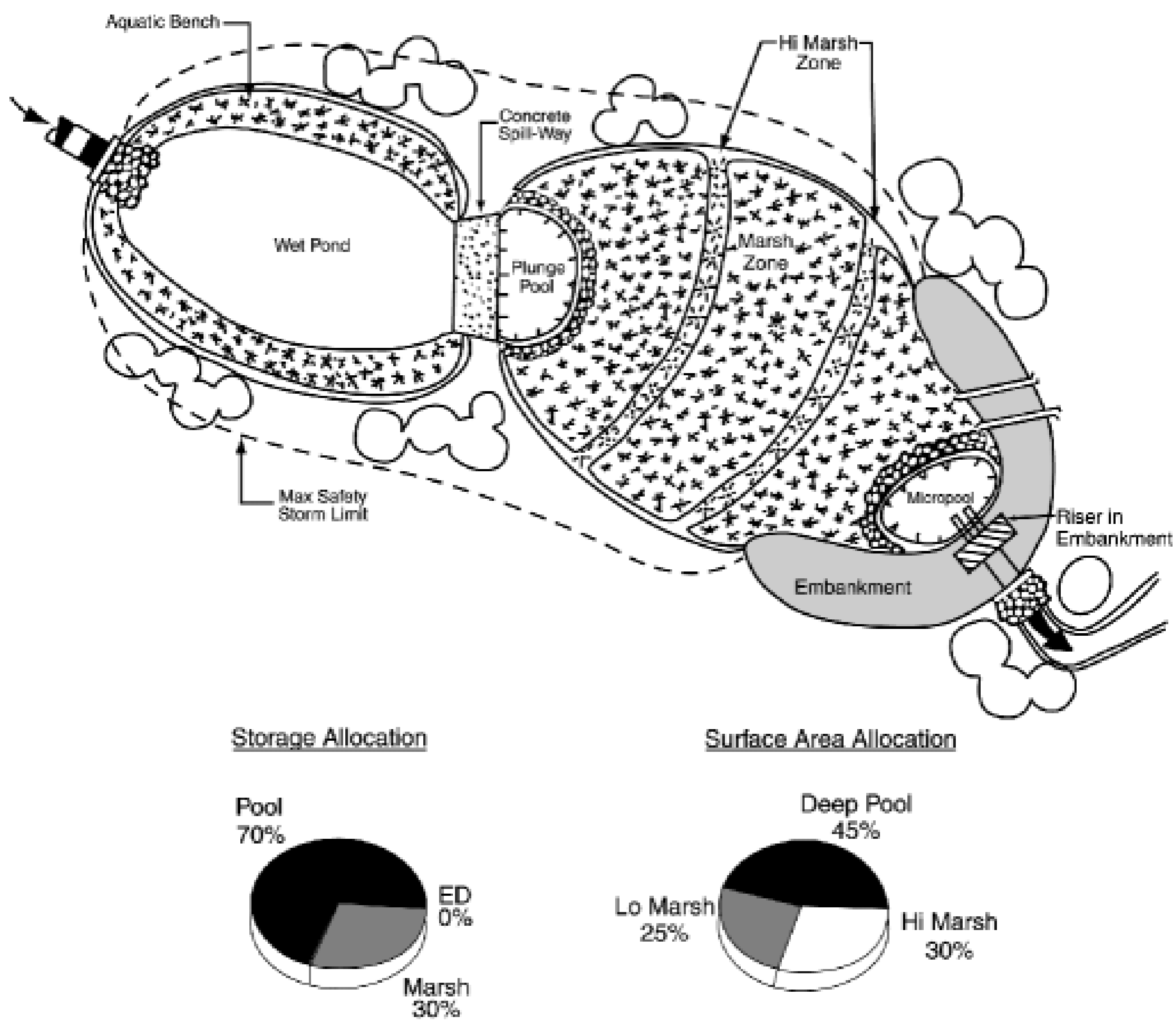


Figure 8.11
Dry Detention Pond Plan and Sections



The pond/wetland system consists of two separate cells - a deep pond leading to a shallow wetland. The pond removes pollutants, and reduces the space required for the system.

Figure 8.12
Stormwater Wetland

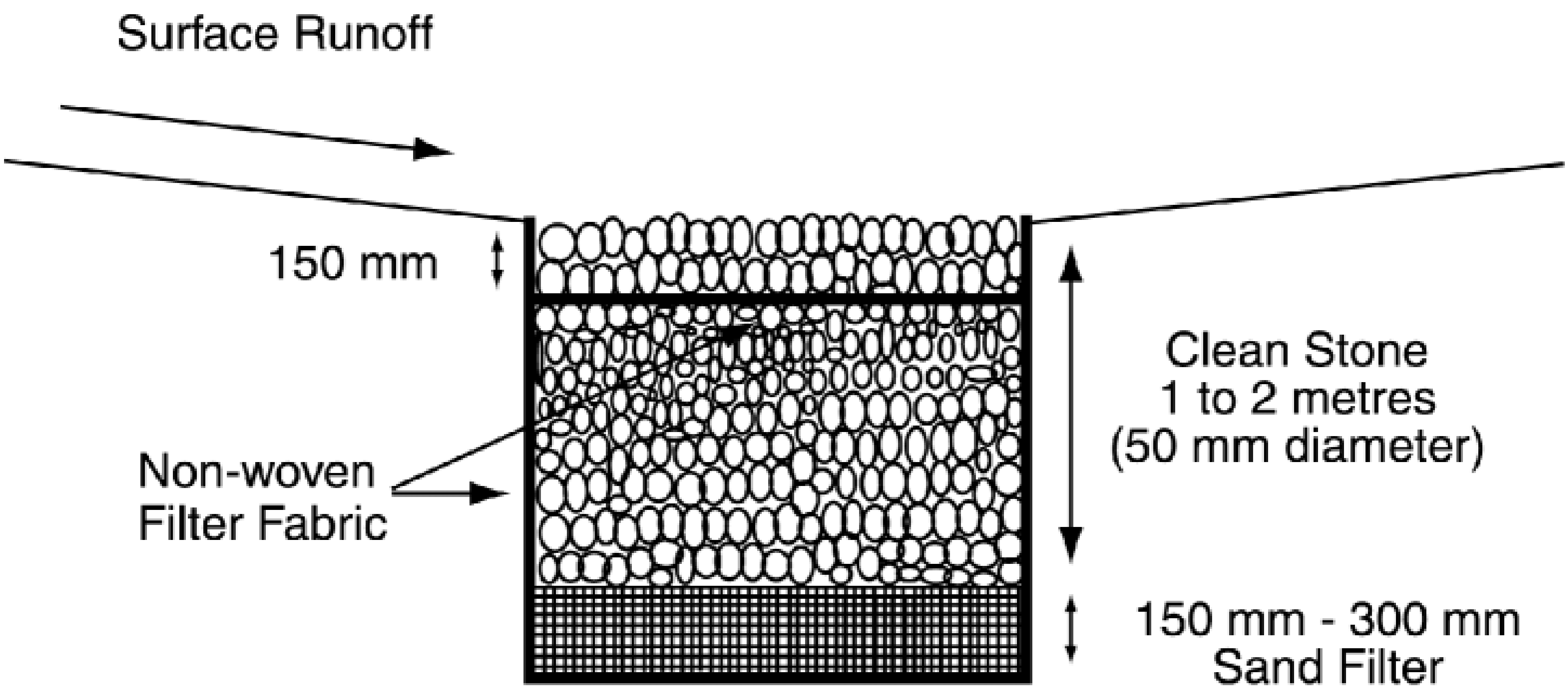


Figure 8.13
Surface Infiltration Trench

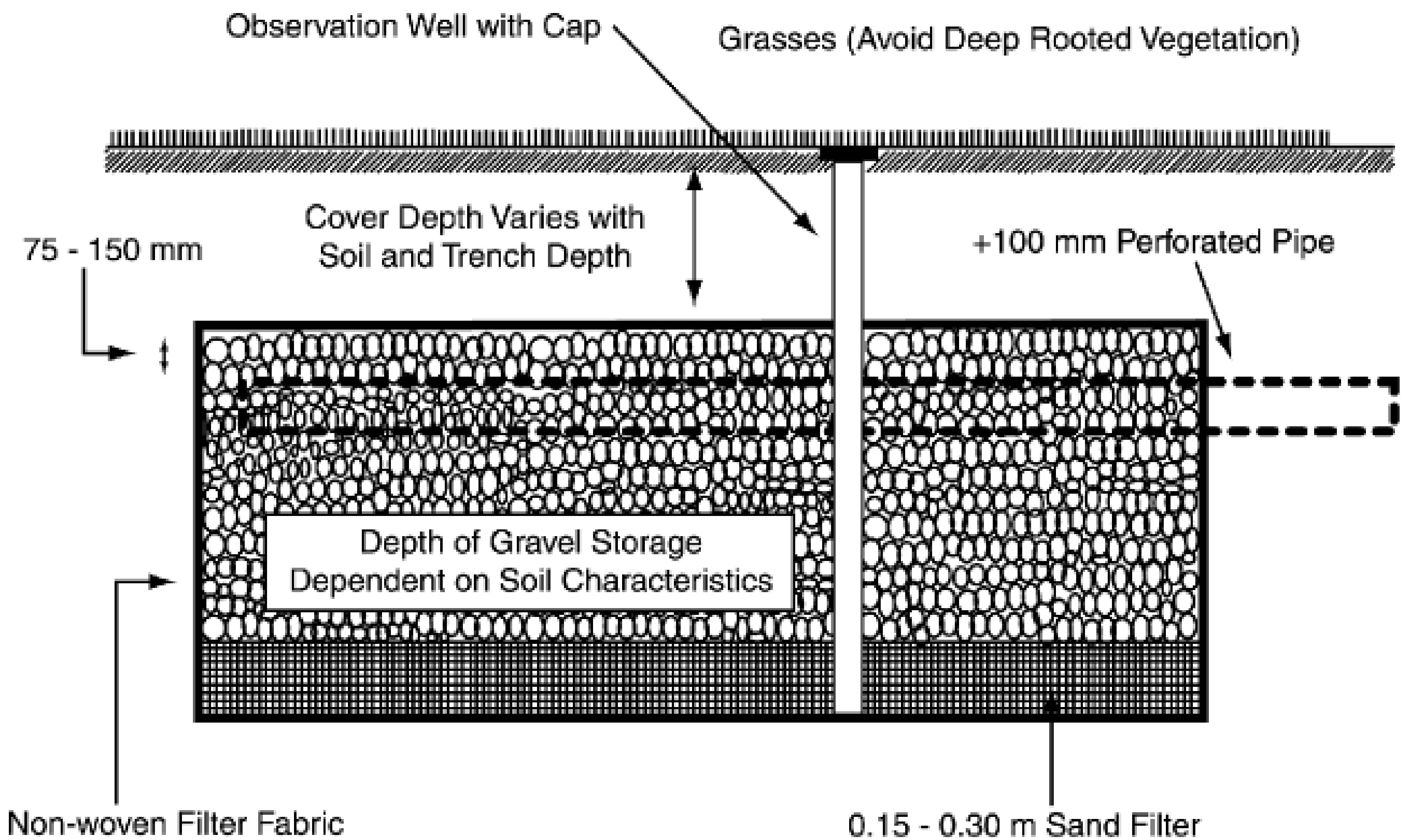


Figure 8.14
Subsurface Infiltration Trench

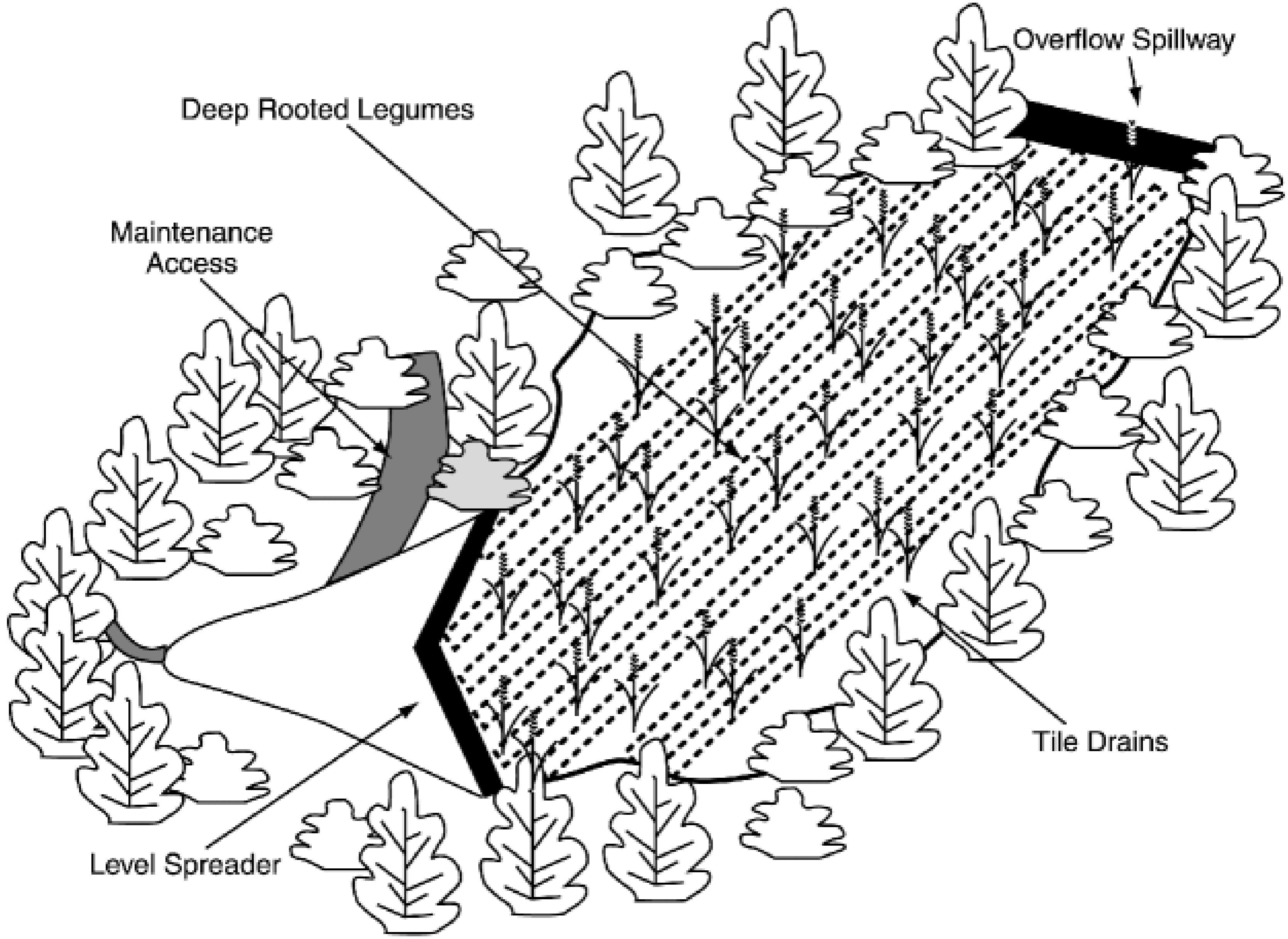


Figure 8.15
Infiltration Basin

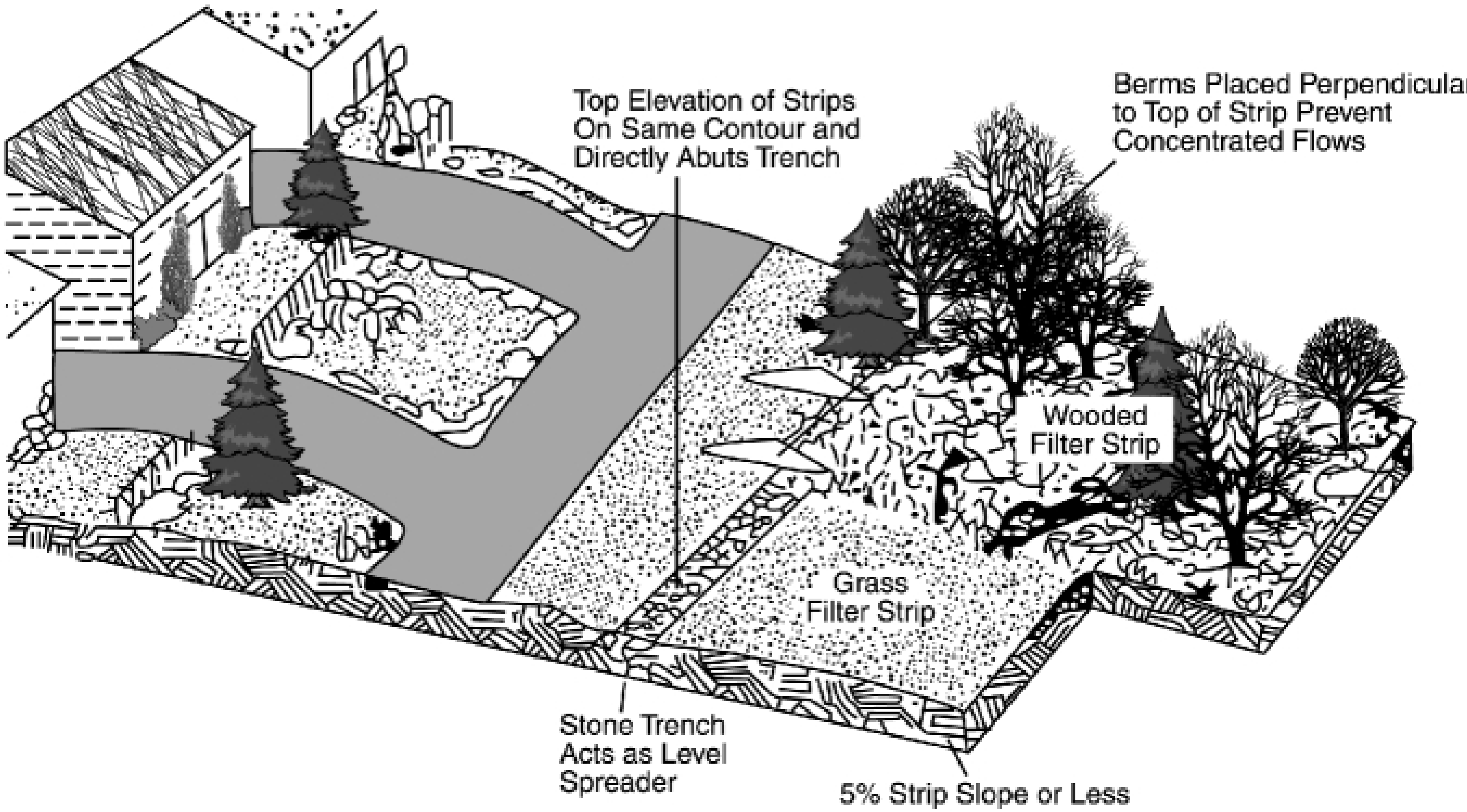


Figure 8.16
Schematic of Grassed and Wooded Filter Strip

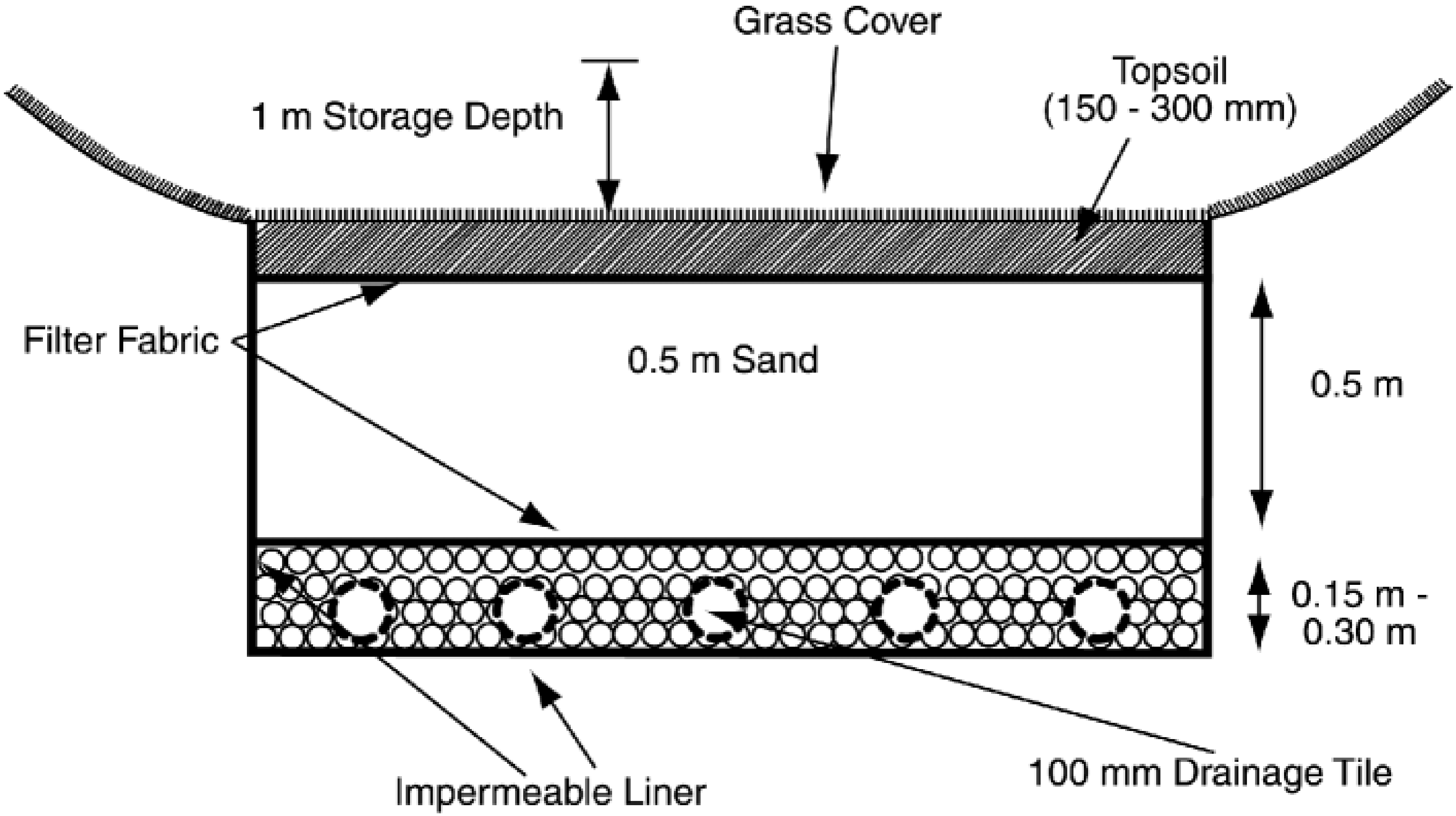


Figure 8.17
Sand Filter Cross Section Profile

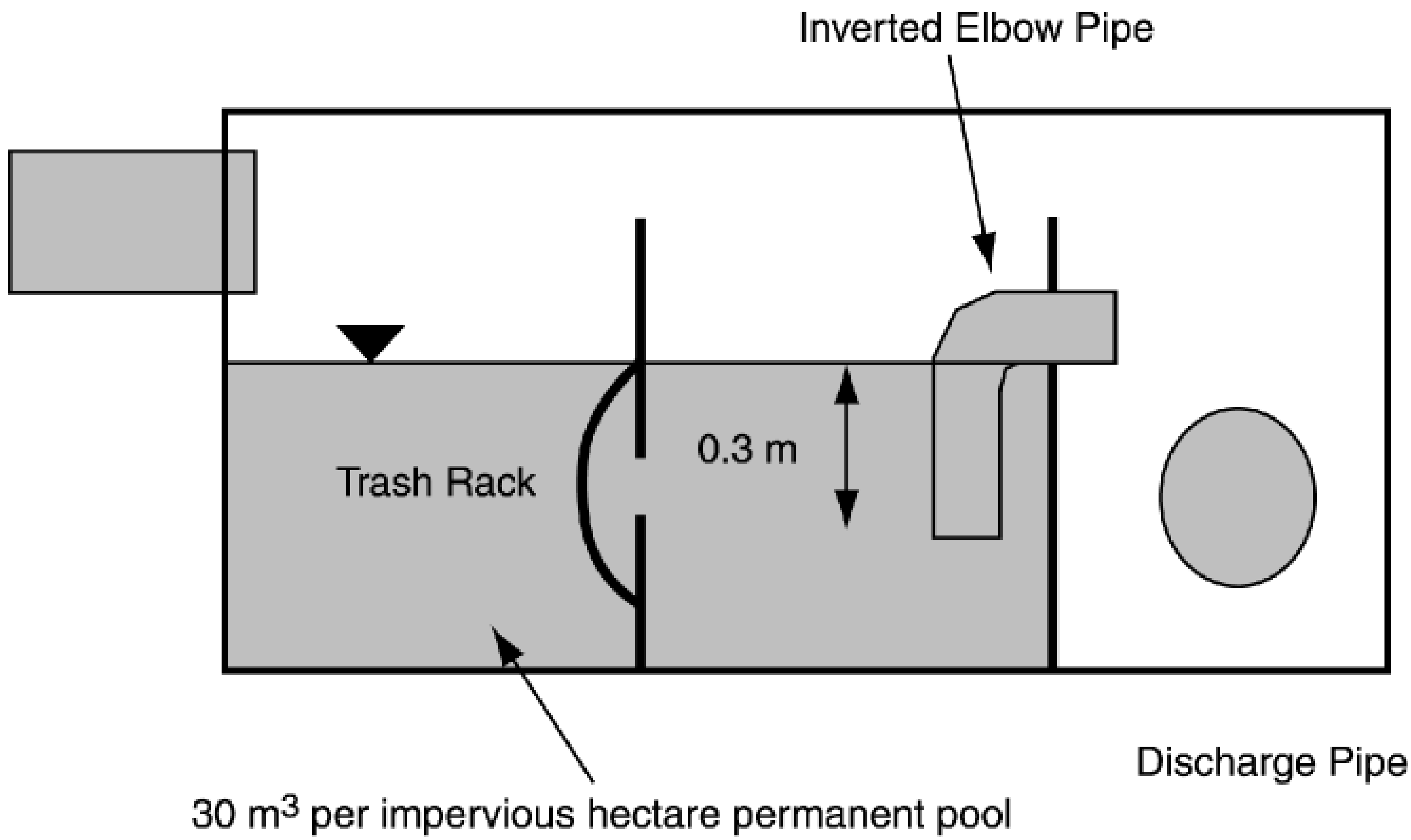


Figure 8.18
Standard 3 Chamber Oil/Grit Separator

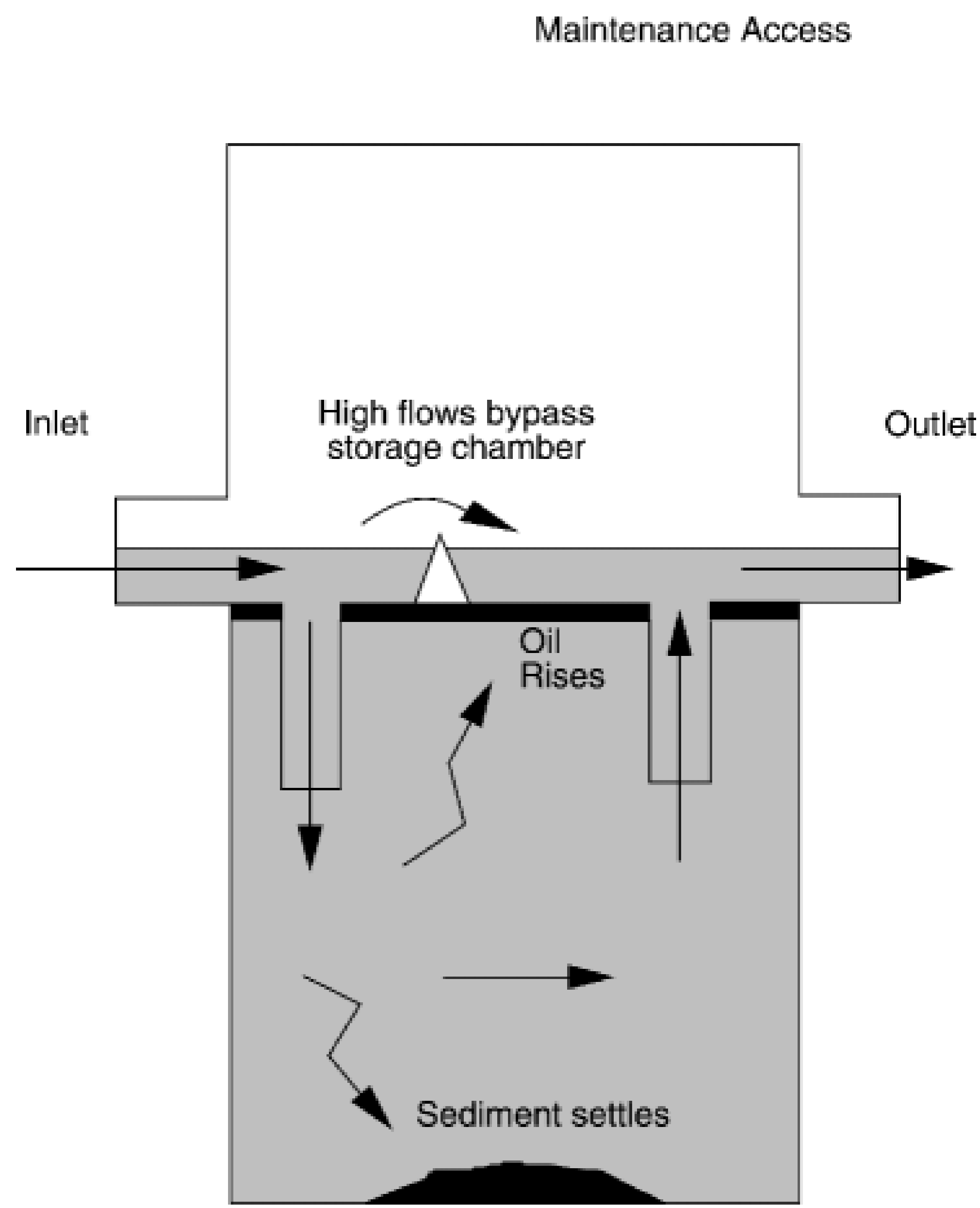


Figure 8.19
Bypass Separator

9.0 OPERATING AND MONITORING REQUIREMENTS AND GUIDELINES - WATERWORKS SYSTEMS

9.1 System Operations

9.1.1 General

The proper operation and maintenance of waterworks system is essential to ensure ongoing sustained production and delivery of the best quality drinking water that is both wholesome and protective of public health. It is therefore imperative that programs and activities such as good operator training, emergency response planning, corrective action measures, cross-connection controls, etc. are in place to ensure a reliable and well operated waterworks system.

9.1.2 Reliability

1. The waterworks system should provide an adequate quantity of safe drinking water in a reliable manner at all times. In determining whether a proposed public water system or an expansion or modification of an existing system is capable of providing an adequate quantity of water, the owner should consider the immediate as well as the reasonably anticipated future needs of the system's consumers.

2. The owner should ensure that the system is operated, maintained and has appropriate backup facilities to protect against failures of the power supply, treatment process, equipment, or structure. Security measures should assure the safety of water source, water treatment processes, water storage facilities and the distribution system.

3. Water pressure at the customer's property line should be maintained at the approved design pressure under maximum hourly design flow conditions. The minimum distribution pressure during peak demand design flow should be 150 kPa.

9.1.3 Operations

1. The waterworks system shall be managed and operated in accordance with the EPEA approval for the facility. The facility shall meet the minimum performance requirements for treatment of components outlined in Sections 2.2 and 2.3 of this document; the treated water shall, at a minimum, meet the health related concentration limits for substances listed in the "Guidelines for Canadian Drinking Water Quality (GCDWQ)."

2. The owner should ensure the development and implementation of an emergency response plan as part of the operations program. The plan should include:

   i) General procedures for routine or major emergencies within the waterworks system; and

   ii) A contingency plan for facilities becoming inoperable in a major emergency.

3. The plant shall be operated within its design capacity to supply treated water.

4. The owner shall not establish nor maintain a bypass to divert water around any feature of a treatment process unless the bypass has been approved by AEP.

5. The owner shall take preventative or corrective action as directed by AEP when results of an inspection conducted by AEP or monthly returns indicate conditions which are currently or may become a detriment to system operations.

6. The owner shall protect waterworks systems from contamination due to cross-connections.

   Further, the owner should develop and implement a cross-connection control program. The scope and complexity of the program should be directly related to the size of the system and the potential public health risk.

   When an existing cross-connection poses a potential health or system hazard, the owner shall shut off water service until the cross-connection has been eliminated or controlled by the installation of a proper backflow prevention assembly.

9.1.4 Facility Classification and Operator Requirements

9.1.4.1 Facility Classification

On recommendation from Water and Wastewater Operator Certification Advisory Committee, Alberta Environmental Protection will classify all waterworks facilities. Facility classification may also be reviewed upon request by the owner or authorized representative. The classification of Water Distribution (WD) systems is based upon the population served by the facilities while the classification of Water Treatment (WT) facilities is based upon a range of points as shown in Tables 9.1(a).

9.1.4.2 Requirement For Having Certified Operators

In accordance with AEPEA, day to day operations of waterworks systems should be supervised by one or more persons who hold a valid certificate of qualification for the type and class of facility concerned. The Approval for each facility will state the required number of certified operators and their required level of certification.

Exempted from **these** requirements are:

1. Hamlets that:

   i) utilize a groundwater source with no treatment for health parameters;
   ii) have less than 15 service connections; or
   iii) have less than three kilometres of water distribution system;

2. "Privately owned developments" as defined in the regulations; and

3. Any other systems as determined by AEP.

TABLE 9.1(a)

WATER TREATMENT PLANTS (WT) CLASSIFICATION

ITEM	POINTS
Size	
Maximum population served, peak day (1 pt/10,000 or part of).	1 - 5
Design flow (avg.day) or peak month's production flow (avg.day), whichever is larger (1 pt/5,000 m^3/day)	1 - 5
Water supply source	
Groundwater	3
Surface Water	5
Variation in raw water quality (slight to extreme)*	0 - 10*
Treatment	
Aeration	2
Packed tower aeration	6
pH adjustment	4
Stability or corrosion control	4
Taste and odour control	4
Colour control	4
Iron or iron/manganese removal (includes filtration)	10
Ion exchange softening	10
Chemical precipitation softening (total process)	20
Solids Contact Clarification (includes coag/floc)	14
Coagulant addition	4
Flocculation	6
Sedimentation	5
Filtration (rapid sand)	10
Filtration (pressure or slow sand)	6
Fluoridation	5
Disinfection	5
On site generation of disinfectant	5
Special processes (not otherwise included)	15
In-plant treatment of plant sludge	6
Laboratory control	
Bacteriological/biological*	0 - 10*
Chemical/physical*	0 - 10*

NOTE: Each unit process should have points assigned only once, i.e. for a plant using oxidation, precipitation and filtration for iron removal, add ten (10) points for the iron removal only and nothing for filtration.

* See Table 9.1(b)

TABLE 9.1(b)

WATER TREATMENT PLANT CLASSIFICATION POINT GUIDE

ITEM	POINTS
Variation in raw water quality	0 - 10
Suggested point values are:	
Little or no variation	0
Raw water quality (other than turbidity) varies enough to require treatment changes approximately 10 percent of the time	2
Raw water quality (turbidity) varies severely enough to require pronounced and/or very frequent treatment changes.	5
Raw water quality subject to periodic serious industrial/municipal/agricultural waste pollution	10
Laboratory Control by plant personnel	
Bacteriological/Biological (complexity)	0 - 10
The key concept is to credit bacti/bio lab work done on-site by plant personnel.	
Suggested point values are:	
Lab work done outside the plant	0
Membrane filter procedures	3
Use of fermentation tubes or any dilution method: fecal coliform determination	5
Biological identification	7
Virus/parasite studies or similarly complex work conducted on site	10
Chemical/Physical (complexity)	0 - 10
The key concept is to credit chemical/physical lab work done on-site by plant personnel.	
Suggested point values are:	
Lab work done outside the plant	0
Push button or colorimetric methods for simple tests such as chlorine residual, pH, - up to	3
Additional procedures such as titration, jar tests, alkalinity, hardness - up to	5
More advanced determinations such as numerous inorganics - up to	7
Highly sophisticated instrumentation such as atomic absorption and gas chromatography	10

Table 9.2 summarizes the classification system. The classification system is based on the "degree of difficulty to operate" a facility. The Alberta system is similar to many used across Canada and the United States

TABLE 9.2

FACILITY CLASSIFICATION SYSTEM

FACILITY	BASED UPON	I	II	III	IV
WD*	Population Served	1500 or fewer	1501-15,000	15,001-50,000	50,001 or more
WT**	Range of Points [Table 9.1a)]	30 or fewer	31-55	56-75	76 or more

Notes: AEP may adjust the classification of a facility if the point system does not reflect the actual complexity of that facility.

WD - Water Distribution
WT - Water Treatment

Water transmission and storage may be part of either water treatment or water distribution, but alone, it is not considered to be either water treatment or water distribution.

* Simple "in-line" treatment (such as shock chlorination) or in-line booster pumping is considered an integral part of the distribution system.

** A groundwater supply with only chlorination is considered a distribution system and not a water treatment plant.

9.1.4.3 Responsibility of Operators

It is the responsibility of certified operators to know and understand the terms and conditions in the operating Approval for their facility. It is also their responsibility to understand the certification requirements for operators of their facilities as indicated by the Approval or by the Certification Guidelines.

It is necessary that the chief operator ensures current certification for operators as required by the Approval or by the Certification Guidelines. It is also important that each facility has a contingency plan so that certified operator requirements are met in cases of planned absences (eg., vacation), unplanned absences (eg., illness), or change of staff (eg., retirement).

Certified operators are also responsible to establish or understand contingency plans for each facility that ensure that the Approval requirements, with respect to certified operators, are met at all times. This is important during normal operation or in the cases of planned absences (eg. vacation), unplanned absences (eg. illness), or change of staff (eg. retirement).

9.1.4.4 Responsibility of Facility Owners

It is the legal responsibility of the owner or manager of each facility to be aware of the requirements of the Approval and to ensure that the requirements are met. The Approval issued by AEP will designate the minimum number and level of certification of key operations personnel. It is important that facility owners or managers develop an internal program so that substitute or replacement personnel are available when necessary.

9.1.4.5 Facility Staffing Requirements: Certified Operators

For Class I facilities, there must be a certified Small Systems or Level I (or higher) operator in charge of the day to day operation of that facility. A back-up certified operator is recommended.

For Class II facilities, there must be a certified Level II (or higher) operator in charge of the day to day operation of that facility. It is recommended that an assistant operator with Level I or II certification be available.

For Class III facilities serving a population under 1,500, there must be a certified Level III (or higher) operator in charge of the day to day operation.

For Class III facilities serving a population under 1,500 - 10,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be at least one other operator certified at Level I or higher.

For Class III facilities serving a population of 10,000 - 50,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be at least one other operator certified at Level II or higher.

For Class III facilities serving a population over 50,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be another operator certified at Level II or higher to act in the absence of the charge operator and at least one other operator certified at Level I or higher. There must be at least one certified operator (any level) for each shift when shift operation is required.

For Class IV facilities serving a population up to 200,000, there must be a Level IV operator in charge of the day to day operation. There must also be two Level III (or higher) operators to act in the absence of the Level IV operator. There must be at least one Level II or higher certified operator for each shift when shift operation is required.

For Class IV facilities serving a population over 200,000, there must be a certified Level IV operator in charge of the day to day operation. There must be at least one other certified Level IV operator to act in the absence of the charge operator. There must be a third operator who is certified at either Level III or IV and there must be at least one Level II (or higher) certified operator for each shift when shift operation is required.

9.2 System Monitoring

9.2.1 General

Establishing reasonable and appropriate monitoring requirements for waterworks facilities is a key factor in ensuring safe drinking water. AEP considers monitoring to fall into one of the following general categories:

1. Operational monitoring
2. Treatment Performance and Compliance monitoring
3. Follow-up or issue oriented monitoring.

Types of monitoring are discussed in detail in the next few sections.

9.2.1.1 Sampling Procedures and Analytical Methods

The owner should ensure that:

1. - Collection and preservation of samples and all analytical procedures are in accordance with the latest edition of "Standard Methods for the Examination of Water and Wastewater," as published by the American Public Health Association, American Water Works Association, and the Water Pollution Control Federation; or

   - by a method outlined in the most recent edition of the "Methods Manual for Chemical Analysis of Water and Wastes" or "Methods Manual for Chemical Analysis of Trace Organics and Pesticides in Environmental Samples," published by Alberta Environmental Protection; or

   - by an alternative method approved by AEP.

2. - Collection, preservation and the analysis of samples are performed by a laboratory approved by AEP.

9.2.1.2 Approval of Analytical Procedures

Owners should ensure that laboratories obtain approval from AEP for the use of any analytical procedures not included in the "Standard Methods."

The laboratory would be required to follow a protocol established by AEP for the approval of analytical procedures not included in the "Standard Methods."

9.2.2 Operational Monitoring

Operational monitoring and associated reporting requirements would be established on a site-specific basis. The nature of the water supply source, the type of treatment system employed and the size/capabilities of the owner are all considered when establishing operational monitoring requirements. Operational monitoring requirements are established both for specific process control purposes, and to ensure that a facility receives good operational attention on a regular basis.

Table 9.3 outlines the operational monitoring requirements that would apply to waterworks systems.

9.2.3 Treatment Performance and Compliance Monitoring

One of the issues created by comprehensive and ever-expanding drinking water quality guidelines is the frequency at which drinking waters should be sampled and which parameters should be analyzed to determine performance of the treatment and compliance with the guidelines. In Alberta, treatment performance and compliance monitoring requirements for municipal waterworks systems are partially established by AEPEA.

The Potable Water Regulation 122/93 requires bacteriological quality monitoring in accordance with the Guidelines for Canadian Drinking Water Quality, and physical, microbiological, radiological and chemical quality monitoring at least once per year for well water supplies and twice per year for surface water supplies. The regulation also indicates that AEP may specify the physical, microbiological, radiological and chemical parameters necessary for the analysis of water samples. This section outlines the specific parameters that have to be monitored, including the sampling location and monitoring frequency.

9.2.3.1 Bacteriological

1. Sampling Location

   Bacteriological samples should be collected from representative points after treatment and throughout the distribution system after the first service connection. Ideally, the owner should prepare a Coliform Monitoring Plan to include at a minimum, a system map or diagram showing the locations of:

   i) Water source
   ii) Storage, treatment and pressure regulation facilities
   iii) Distribution system
   iv) Pressure zones
   v) Coliform sample collection sites

   The owner should revise or expand the plan at any time the plan no longer ensures representative monitoring of the system, and keep the Coliform Monitoring Plan on file and make it available to AEP for inspection upon request.

2. Monitoring Frequency

   The number of required routine coliform samples to determine compliance should be in accordance with the GCDWQ, or as directed by AEP.

3. Invalid Samples

   When a coliform sample is determined invalid by the laboratory, the municipality should:

   i) Not include the sample in the determination of monitoring compliance; and
   ii) Collect and submit for coliform analysis, an additional drinking water sample from the same location as each invalid sample within twenty-four hours of notification by the laboratory or AEP.

4. Compliance Criteria

   Compliance criteria for bacteriological quality shall be in accordance with GCDWQ.

9.2.3.2 Inorganic Chemical and Physical

1. Parameters to be monitored

   A complete inorganic chemical and physical analysis should consist of the primary and secondary inorganic chemical and physical substances.

   i) The primary inorganic chemical and physical substances are those substances with MACs in the GCDWQ and which are known to cause adverse effects on health. Primary chemical and physical substances are arsenic, barium, boron, cadmium, chromium, cyanide, fluoride, lead, mercury, nitrate and nitrite (as N), selenium, and turbidity.

   ii) The secondary inorganic chemical and physical substances are those substances with AOs in the GCDWQ with limits below those considered to constitute a health hazard; and the parameters without guidelines identified in the GCDWQ. The secondary chemical and physical substances are ammonia, asbestos, calcium, chloride, colour, copper, hardness, iron, magnesium, manganese, pH, silver, sulphate, total dissolved solids, total organic carbon and zinc.

   For specific systems, AEP, at its discretion, may revise the list of primary and secondary inorganic chemicals and physical substances to be monitored.

2. Sampling Location

   i) Inorganic chemical and physical samples should be collected from a point representative of each source, after treatment, and prior to entry to the distribution system. The point of collection should be designated as "Sampling Location" and confirmed by AEP.

TABLE 9.3

OPERATIONAL MONITORING REQUIREMENTS

Parameter	Point of Measurement	Requirement/Objective	Minimum Monitoring
Raw water turbidity	Before addition of any chemical or treatment process	None	See Section 9.2.3.3
Treated water turbidity	Immediately after filtration before entering the clearwater tank	See Section 2.2.1(1)	See Section 9.2.3.3
Raw water flows	Entering the treatment plant	Not to exceed treatment plant's design capacity	Once per day for total daily flow
Treated water flows	Entering the clearwater tank or the distribution system	None	Once per day for total daily flow
Raw water Giardia levels	Entering the treatment plant	None	Quarterly
Raw water pH	Before addition of any chemical	None	Once per day using grab sampling
Raw water iron and manganese	Before addition of any chemical	None	Once per week using grab sampling
Chemicals used	Feed point	The chemical dosage should not exceed the recommended maximum concentration authorized by AEP	Volume/Weight/ Concentration of chemicals used daily or weekly
Treated water pH	Entering the distribution system	6.5-8.5 (exceptions are acceptable if disinfection is not compromised)	Once per day using grab sampling
Turbidity within distribution system	Random location throughout the distribution system	Not to exceed 5 NTU	Once per week using grab sampling
Treated water iron	Immediately after filtration before entering the clearwater tank	Not to exceed 0.3 mg/l	Five times per week, twenty-four hours apart using grab sampling
Treated water manganese	Immediately after filtration before entering the clearwater tank	Not to exceed 0.05 mg/l	Five times per week, twenty-four hours apart using grab sampling

Notes:
1. Monitoring of these parameters is not required during the calendar days the treatment plant is not operated or during statutory or civic holidays.
2. Raw and treated water turbidity monitoring are also required from a compliance point of view.
3. Raw water Giardia levels will be based on the geometric mean of a minimum of 4 quarterly samples, or as determined by AEP
4. Raw water flow rates should be reported in m^3/s and the total daily flows should be reported in m^3/d.
5. Specified monitoring for iron and manganese is required only for plants treating for these parameters.

ii) For multiple sources or wellfields within a single system, which are blended prior to entry into the distribution system, the owner may identify an "Alternate Sampling Location" based on the following:

a) Source vulnerability;
b) Individual source characteristics;
c) Previous water quality information; and
d) Any other information

Each sample must be taken at the same point unless conditions make another sampling point more representative of the water produced by the treatment plant.

iii) When the owner provides treatment for one or more inorganic chemical and physical contaminants, AEP may require the owner to sample before and after treatment. The "Source Sampling Location" for raw water supply should be confirmed by AEP. For groundwater supply requiring treatment, each well should be sampled at the source.

3. Monitoring Frequency

i) The frequency of monitoring conducted to determine compliance with the MACs and AOs for the primary and secondary inorganic chemical and physical substances respectively, should be once per year for groundwater supplies and twice per year, once in the summer and once in the winter for surface water supplies at each designated "Sampling Location" or "Alternate Sampling Location."

For groundwater requiring treatment, one additional sample would be required at each "Source Sampling Location."

ii) Where the results of sampling for the primary inorganic chemical and physical substances indicate that MACs have been exceeded, AEP would require that one additional confirmation sample be collected as soon as possible (but not to exceed two weeks) after the initial sample results are received.

iii) Systems which exceed the MACs for the primary inorganic chemical and physical substances in confirmation samples, should monitor quarterly beginning in the next quarter after the violation occurred, or as directed by AEP. The owner may revert back to the frequencies specified in sub-section 9.2.3.2(3)(i) above, provided that the system is reliably and consistently producing water below the MACs. AEP would make this determination based on a minimum of two quarterly samples for groundwater systems, and a minimum of four quarterly samples for surface water systems.

4. Compliance

i) For systems which are monitoring annually, the system is out of compliance, if the level of a substance at any sampling point is greater than the MAC. If a confirmation sample is required by AEP, the determination of compliance will be based on the average of the two samples.

ii) For systems which are conducting monitoring at a frequency greater than annual, compliance with the MACs is determined by a running annual average at any sampling point. If the average at any sampling point is greater than the MAC, then the system is out of compliance. Any sample below the detection limit should be calculated at zero for the purpose of determining the annual average.

9.2.3.3 Turbidity

1. Source Turbidity Monitoring

   i) Owners on surface water systems using conventional, direct or in-line filtration should measure source turbidity at least once per day on a representative sample collected before the addition of any chemicals.

   ii) Grab sampling or continuous turbidity monitoring and recording may be used to meet the requirement specified in (i) of this sub-section.

   iii) Owners who measure turbidity continuously should record measurements at equal intervals, at least every four hours. Daily turbidity would be the arithmetic average of all the turbidity measurements in one calendar day.

2. Treated Water Turbidity Monitoring

   i) The owners should:

      a) continuously monitor turbidity on representative samples on each individual filter effluent and on the system's combined filter effluent, prior to clearwater tank; and

      b) record continuous turbidity measurements at equal intervals, at least every four hours. Daily turbidity would be the arithmetic average of all the turbidity measurements in one calendar day.

   ii) Municipalities using slow sand filtration may reduce treated water turbidity monitoring to one grab sample per day with AEP approval. Reduced turbidity monitoring would be allowed only if the owner can demonstrate that a reduction in monitoring will not endanger the health of consumers served by the water system.

3. Validation

   Municipalities that continuously monitor turbidity using an in-line analyzer must establish a QA/QC program, consisting of analytical procedures in the "Standard Methods," to validate the measurements obtained from continuous monitoring.

4. Compliance

   System compliance for turbidity is detailed under the Performance Standards in Section 2.2.1(1).

9.2.3.4 Fluoride

1. Where fluoridation is practiced, the municipalities should:

   i) Measure the raw water fluoride concentration, at least once a week on a representative sample collected before the addition of any chemicals. A grab sample may be used for this purpose;

   ii) Once a month, collect a representative sample of raw water before the addition of any chemical and forward it to an independent laboratory for the measurement of fluoride concentration;

   iii) Measure the treated water fluoride concentration continuously at equal intervals, every four hours or at least once per day on a representative sample entering the distribution system;

   iv) Once a month, collect a representative sample of treated water entering the distribution system and forward it to an independent laboratory for the measurement of fluoride concentration;

   v) Measure the total daily volume of water to which fluoride is added;

   vi) Measure the total daily weight of fluoride added to the water;

   vii) Measure the total daily weight of fluoride in the feeding equipment; and

   viii) Measure the total daily weight of fluoride in stock.

2. Validation

   See section 9.2.3.3(3)

3. Compliance

   System compliance for fluoride ion concentration in drinking water is outlined under the Performance Standards in Section 2.5.

9.2.3.5 Trihalomethanes

1. Sampling Location and Monitoring Frequency

   i) Owners on a surface water source, serving a population of ten thousand or more and providing water treated with chlorine should monitor as follows:

      a) Owners should collect four samples per treatment plant every three months. The samples should be taken within a twenty-four hour period. One of the samples should be taken from the extreme end of the distribution system and three samples from representative locations in the distribution system. The samples should be analyzed for total trihalomethanes (TTHM). After one year of monitoring, AEP may reduce the monitoring frequency to one sample every three months per treatment plant if the TTHM levels are less than 100 µg/L.

The sample should be taken at the extreme end of the distribution system.

b) Owners on Regional Systems should collect one water sample every six months. The sample should be taken at the extreme end of the distribution system and analyzed for TTHM.

ii) Owners on a surface water source, serving a population of less than ten thousand, and providing water treated with chlorine should monitor as follows:

a) Owners should collect one water sample per treatment plant every three months for one year. The sample should be taken at the extreme end of the distribution system and analyzed for TTHM. After the first year, the monitoring may be reduced to once every three years, if TTHM levels are less than 100 μg/L.

b) Owners on Regional Systems should collect one water sample every three months for one year at the extreme end of the distribution system and analyze for TTHM. After the first year, the monitoring may be reduced to once every three years, if TTHM levels are less than 100 μg/L.

iii) TTHM monitoring requirements for groundwater systems will be determined on a site specific basis.

2. Compliance

Compliance with the MAC is determined by a running average of all samples taken during any twelve month period. If the average exceeds the MAC, then the system is out of compliance.

9.2.3.6 Organic Chemicals and Pesticides

1. Parameters to be Monitored

Municipalities serving a population of ten thousand or more would undertake monitoring for organic chemicals and pesticides. Parameters to be monitored should include all the organic chemicals and pesticides (used in Alberta only) that are listed in the GCDWQ. They are:

atrazine, benzene, benzo(a)pyrene, bromoxynil, carbaryl, carbon tetrachloride, chlorpyrifos, cyanazine, diazinon, dicamba, dichlorobenzene 1, 2-, dichlorobenzene 1.4-, dichlorethane 1, 2-, dichloromethane, dichlorphenol, 2, 4-, 2, 4-D, demethoate, ethylbenzene, glyphosate, lindane, malathion, monochlorobenzene, pentachlorophenol, phorate, picloram, tetrachlorophenol 2, 3, 4, 6-, toluene, triallate, trichloroethylene, trichlorophenol 2, 4, 6- and trifluralin.

The need for organic chemicals and pesticide monitoring by municipalities serving a population of less than ten thousand will be determined by AEP on a site-specific basis.

For specific systems, AEP, at its discretion, may revise the list of organic chemicals and pesticides to be monitored.

2. Sampling Location

Sampling locations for organic chemicals and pesticides should be the same locations as for inorganic chemical and physical samples. See Section 9.2.3.2(2) for details.

3. Monitoring Frequency

i) The frequency of monitoring conducted to determine compliance with the MACs for organic chemicals and pesticides should be once per year in the summer for groundwater supplies, and twice per year, once in early spring (March-April) and once in the summer (May-September) for surface water supplies, at each designated "Sampling Location" or "Alternate Sampling Location."

For groundwater requiring treatment, one additional sampling would be required at the "Source Sampling Location."

ii) Where the results of sampling for the parameters in subsection 9.2.3.6(1) indicate that MACs have been exceeded, AEP will require that one additional confirmation sample be collected, as soon as possible (but not to exceed two weeks) after the initial sample results are received.

iii) Systems which exceed the MACs in confirmation samples should monitor quarterly at each sampling point which resulted in violation, or as directed by AEP. AEP may decrease the quarterly monitoring requirement to what is specified in subsection 9.2.3.6(3)(i), provided it has determined that the system is reliably and consistently below the MACs. This determination will be based on a minimum of two quarterly samples for groundwater systems, and a minimum of four quarterly samples for surface water systems.

iv) If monitoring shows that the system is reliably and consistently below the MACs for three consecutive years, AEP may allow surface water systems to be monitored annually, and groundwater systems to be monitored once in three years.

4. Compliance

Compliance requirements for organic chemicals and pesticides are the same as for inorganic chemical and physical parameters, as detailed in section 9.2.3.2(4).

9.2.3.7 Disinfection (See Also Section 4.1.5.1)

1. Determination of disinfectant contact time (T_{10})

i) The owner should calculate T_{10} at maximum hourly flow.

ii) For pipelines, T_{10} is calculated by dividing the internal volume of the pipe by the maximum hourly flow rate through that pipe.

iii) For all other systems components, tracer studies or empirical methods should be used to determine T_{10}.

iv) The owner should use the T_{10} value determined by tracer studies or other methods as T on all CT Calculations.

v) Tracer studies

a) The owner should conduct field tracer studies on all systems components for which similar contact times are not documented.

b) Ideally, three tracer studies should be done for different flow conditions at various depths of clearwater tanks.

c) The tracer studies should be conducted in accordance with good engineering practices using methods acceptable to AEP.

vi) Empirical Methods

Empirical methods may be used to calculate T_{10}, if the owner can demonstrate that system components have configuration similar to components on which tracer studies have been conducted. See Appendix C for illustration of typical baffling conditions in reservoirs.

2. Establishing the Level of Reduction

i) AEP will establish the level of disinfection (log reduction) to be provided by the municipality.

ii) The required level of reduction will be based on source quality and expected levels of <u>Giardia</u> cyst and virus reduction achieved by the systems filtration process. Regardless of the reduction credit allowed for filtration, minimum requirements for disinfection alone would be 0.5 log reduction of <u>Giardia</u> cysts and 2 log reduction of viruses.

iii) Based on periodic review, AEP may adjust, as necessary, the level of disinfection the owner should provide to protect the public health.

iv) For systems not meeting the turbidity requirements outlined in Sections 2.2.1(1) and 2.2.2, AEP may grant reduced filtration credit or no filtration credit, as per Section 2.2.4(3).

3. Monitoring the Level of Reduction and Removal

i) Each day the system is in operation, the municipality should determine the total level of reduction of <u>Giardia</u> cysts and viruses.

ii) The owner should determine the total level of reduction based on:

a) <u>Giardia</u> cyst and virus reduction credit granted by AEP for filtration; and

b) Level of reduction of <u>Giardia</u> cysts and viruses achieved through disinfection.

iii) At least once per day, the owner should monitor the following to determine the level of reduction achieved through disinfection:

a) Temperature of the disinfected water at each residual disinfectant concentration sampling point used for CT calculations; and

b) If using chlorine, pH of the disinfected water at each chlorine residual disinfectant concentration sampling point used for CT calculations.

iv) Each day during peak hourly flow (based on historical information), the owner should:

a) Ascertain the filled capacity/depth of the clearwater tank;

b) Determine the disinfectant contact time, T, based on clearwater tank capacity/depth, to the point at which C is measured.

v) The owner should measure the disinfectant concentration, C, of the water at the point for which T is calculated.

For systems serving more than five thousand (>5000) people, the owner should continuously monitor, at equal intervals and at least ever four hours, and record the residual disinfectant concentration. The lowest recorded value for C should be used in the CT calculations.

For systems serving less than five thousand (<5000) people, a grab sample may be collected at the maximum hourly flow to determine the C for CT calculations.

The C measurement point should be located before or at the first customer.

vi) Validation

See section 9.2.3.3(3)

4. Determining the Level of Reduction

Each day the system serves water to the public, the owner should determine:

The total reduction ratio $\frac{(CT_{actual.})}{(CT_{required})}$

CT_{actual} values should be determined using the monitored values of C and T as outlined in subsection 9.2.3.7(3). If C and T values are monitored more frequently, the lowest CT values calculated should be used in determining the level of reduction. $CT_{required}$ values should be referenced from Appendix A or Appendix B.

5. Determining Compliance with the Required Level of Reduction

   The system will be considered in compliance with the reduction requirement when the total reduction ratio is greater than 1. The reduction ratio may be less than 1 for a maximum of one day a month.

6. Monitoring the residual disinfectant concentration entering the distribution system, at a point immediately downstream of the clearwater tank.

   i) Systems serving more than five thousand (>5000) people.

      a) The owner should continuously monitor and record the residual disinfectant concentration of water entering the distribution system and report the lowest value each day.

      b) If the continuous monitoring equipment fails, the owner should measure the residual disinfectant concentration on grab samples collected at least every four hours at the entry to the distribution system.

   ii) Systems serving five thousand or less ($\leq$5000) people.

      a) The owner should collect grab samples or use continuous monitoring and recording to measure the residual disinfectant concentration entering the distribution system.

      b) Owners choosing to take grab samples collect:

         - Samples at the following minimum frequencies

Population Served	Number/day
< 500	1
501 - 1,000	2
1,001 - 2,500	3
2,501 - 5,000	4

         - At least one of the grab samples at peak hourly flow; and the remaining samples evenly spaced over the time the system is disinfecting water that will be delivered to the public.

7. Monitoring residual disinfectant concentrations within the distribution system.

   i) The owner should measure the residual disinfectant concentration at representative points within the distribution system once daily or as otherwise approved by AEP.

   ii) At a minimum, the owner should measure the residual disinfectant concentration within the distribution system at the same time and location that a routine or repeat coliform sample is collected.

8. Determining compliance with the required level of residual disinfectant.

   See section 2.2.3 for compliance with residual disinfectant requirements.

9.2.4 Issue Oriented and Follow-Up Monitoring

1. General

   i) Follow-up action by the owner may be required when the system does not meet the minimum potable water quality stipulated in Section 2.1 or the minimum performance requirements for treatment stipulated in Sections 2.2 and 2.3. Follow-up and corrective actions for specific parameters are discussed later in this section.

   ii) When a violation of MAC or minimum performance requirements for treatment occurs, the municipality should:

      a) Notify AEP in accordance with section 9.4;

      b) Determine the cause of the contamination or operational problems; and

      c) Take action as directed by AEP.

2. Bacteriological

   i) When coliform bacteria are present in any sample, or if a sample contains either more than 500 HPC colonies per millilitre, or more than 200 background colonies on a total coliform membrane filter, the municipality should ensure that the following actions are taken:

      a) The sample is analyzed for fecal coliform or E.coli;

      b) Repeat samples are collected in accordance with (ii) of this subsection; and

      c) The cause of the coliform/colonies presence is determined and corrected.

   ii) Repeat Samples

      a) The owner should collect and submit for analysis a set of repeat samples (consisting of three repeat samples for every sample in which the presence of coliform/colonies is detected.

      b) The three repeat samples should be collected as follows:

         - At the site of previous sample with a coliform/colonies presence;
         - Within 5 active services upstream of the site of sample with a coliform/colonies presence; and
         - Within 5 active services downstream of the site of sample with a coliform/colonies presence.

c) All samples in a set of repeat samples should be collected on the same day of notification by the laboratory of a coliform/colonies presence, and submitted for analysis within twenty-four hours.

d) When repeat samples have coliform/colonies present, the owner should contact AEP and collect one additional set of repeat samples for each sample where a coliform/colonies presence was detected. The procedure should be continued until the problem is corrected.

iii) Corrective Actions

If coliform/colonies are detected in treated water, corrective action should be taken immediately in consultation with AEP, and should as a minimum include:

a) Increasing disinfectant dosage

b) Flushing water mains

c) Using an alternative source.

3. Disinfectant

i) When the total inactivation ratio is less than 1, the owner should:

a) Stop water production until the ratio is restored back to 1;

b) Notify AEP in accordance with section 9.4; and

c) Undertake corrective actions established in consultation with AEP.

ii) If the residual disinfectant concentration entering the distribution system, measured as free or combined chlorine, is less than 0.2 mg/L, the owner should:

a) Take immediate actions(usually increasing disinfectant dosage or cleaning clearwater tank);

b) Increase the monitoring frequency until the residual meets the specified limit; and

c) Notify AEP in accordance with section 9.4.

iii) If the residual disinfectant concentration within the distribution system is less than 0.05 mg/L, the owner should:

a) Take immediate actions (usually flushing) to obtain the residual; and

b) Increase the monitoring frequency until the residual is detectable.

4. Fluoride

   i) The fluoridation system should be shut down if the owner is unable to test fluoride concentrations.

   ii) If the daily fluoride residual varies outside the range of 0.8 mg/L +/- 0.2 mg/L, the owner should:

      a) Resample and check calculated dosage;

      b) Adjust and recalibrate the feed rate; and

      c) Take a sample to verify that proper fluoride residual levels have been obtained.

   iii) If the fluoride residual levels exceed 1.5 mg/L or if the fluoride residual levels vary 0.8 mg/L +/- 0.2 mg/L on two consecutive days, the owner should:

      a) Resample and check calculated dosage;

      b) Notify AEP in accordance with Section 9.4; and

      b) Undertake corrective actions established in consultation with AEP.

5. Turbidity

   i) If treated water turbidity entering the clearwater tank exceeds 0.5 NTU, the owner should:

      a) Notify AEP in accordance with section 9.4; and

      b) Undertake corrective actions established in consultation with AEP.

6. Inorganic physical, organic, and pesticides

   Inorganic, physical, organic and pesticides follow-up monitoring should be conducted in accordance with the procedures outlined in section 9.2.3.2 and 9.2.3.6.

9.3 Record Keeping

All records should bear the signature of the operator in responsible charge of the water system or his or her representative. Owners should keep these records available for inspection by AEP and should send the records to AEP if requested.

The owners should keep the following records and water quality analyses:

1. Bacteriological and turbidity analysis results should be kept for five years.

2. Chemical analysis results should be kept for as long as the system is in operation.

3. Records of daily source meter readings should be kept for ten years.

4. Other records of operation and analyses required by AEP should be kept for three years.

5. Actual laboratory reports may be kept or data may be transferred to tabular summaries, provided the following information is included:

   i) The date, place and time of sampling, and the name of the person collecting the sample;

   ii) Identification of the sample type (routine distribution system sample, repeat sample, source or treated water sample, or other special purpose sample);

   iii) Date of analysis;

   iv) Laboratory and person responsible for performing analysis;

   v) The analytical method used; and

   vi) The results of the analysis.

6. Records of action taken by the system to correct violations of drinking water standards (MAC), including names and addresses of persons who discovered the contravention. For each violation, copies of public notifications should be kept for three years after the last corrective action taken.

7. Copies of project reports, construction documents and related drawings, inspection reports and approvals should be kept for the life of the facility.

8. Where applicable, daily records including:

   i) Information on level of inactivation of <u>Giardia</u> cysts and viruses achieved through disinfection:

      a) Temperature at each residual concentration sampling point;
      b) pH if using chlorine;
      c) peak flow;
      d) Filled capacity/depth of clearwater tank;
      e) Disinfectant contact time T, and corresponding concentration C;
      f) Inactivation ratio.

   ii) Residual disinfectant concentration entering the distribution system, and at representative points within the distribution system;

   iii) Fluoride level;

   iv) Water treatment plant performance including, but not limited to:

      - Type of chemicals used and quantity;
      - Amount of water treated; and
      - Results of analyses.

v) Turbidity;

vi) Source meter readings; and

vii) Other information as specified by AEP.

All records referred to in this section should be made available at any time upon the request of an inspection or investigator as appointed under DPEA.

9.4 Reporting

1. Reporting requirements shall be in accordance with the operating approval for that facility, issued by AEP.

2. Unless otherwise specified in this section, the owner should report to AEP within forty-eight hours:

   i) The violation of a MAC

   ii) The failure to comply with minimum performance requirements for treatment; and

   iii) The failure to comply with the compliance monitoring requirements.

3. The owner should notify AEP when:

   i) Coliform is present in a sample, within ten days of notification by the laboratory;

   ii) A coliform MAC violation is determined in a sample, within twenty-four hours of determining the violation; and

   iii) Fecal coliform or E. coli is present in a sample, by the end of the business day in which the municipality is notified by the laboratory.

4. On discovering the following contravention, the owner should notify AEP by the end of the business day or during the next working day if discovered after normal close of business day:

   i) when the total disinfectant inactivation ratio is less than 1;

   ii) when treated water turbidity exceeds the specified limits;

   iii) when residual disinfectant concentration entering the distribution system, measured as free or combined chlorine is below 0.2 mg/L;

   iv) when the fluoride residual levels exceed 1.5 mg/L or when the fluoride residual levels vary outside 0.8 mg/L +/- 0.2 mg/L on two consecutive days; and

   v) when the application of fluoride is discontinued in order to repair or replace equipment.

5. The owner should compile and submit an annual report on or before February 28 of the following year in which the information was collected. The report should include the following:

   i) A monthly summary of all operational and compliance monitoring for that particular facility, as identified by AEP;

   ii) A summary of approval contraventions and remedial measures taken; and

   iii) A summary of any permanent upgrading works undertaken during the year.

10.0 OPERATING AND MONITORING REQUIREMENTS AND GUIDELINES - WASTEWATER SYSTEMS

10.1 Systems Operations

10.1.1 General

The proper operation and maintenance of wastewater systems is essential to produce highest quality of treated effluent and to ensure the protection of public health and the environment. It is therefore important that programs and activities such as good operator training, emergency response planning, corrective action measures, etc. are in place to ensure a reliable and well operated wastewater system.

10.1.2 Reliability

1. The wastewater system should produce effluent to meet the required limits at all times. Consideration should be given to optimize operation of the system to handle both dry weather and wet weather flows.

2. The owner should ensure that the system is operated, maintained and has appropriate backup facilities to protect against failures of the power supply, treatment process, equipment, or structure.

10.1.3 Operations

1. The wastewater systems should be managed and operated in accordance with the EPEA approval of the systems. The wastewater treatment facilities should be operated to produce effluent that meet the standards detailed in tables 3.1 and 3.2.

2. Non-domestic discharges should not interfere with the operation of the treatment plant, nor should it impact on the treatability of the wastewater and affect the performance of the plant.

3. The owner should ensure the development and implementation of an emergency response plan as part of the operations program, for emergencies such as pipeline breakage or accidental spills of any toxins to sewers and/or treatment plant. The plan should include:

   i) General procedures for routine or major emergencies within the wastewater system; and

   ii) A contingency plan for facilities becoming inoperable in a major emergency.

4. The plant should be operated within its design capacity.

5. The owner should take preventative or corrective action as directed by AEP when results of an inspection conducted by AEP or monthly returns indicate conditions which are currently or may become a detriment to system operations.

10.1.4 Facility Classification And Operator Requirements

10.1.4.1 Facility Classification

On recommendation from Water and Wastewater Operators Certification Advisory Committee, AEP will classify all wastewater facilities. Facility classification may also be reviewed upon request by the owner or authorized representative. The classification of Wastewater Collection (WWC) system is based upon the population served by the facilities while the classification of Wastewater Treatment (WWT) facilities is based upon a range of points as shown in Tables 10.1(a).

Table 10.2 summarizes the classification system. The classification system is based on the "degree of difficulty to operate" a facility. The Alberta system is similar to many used across Canada and the United States.

10.1.4.2 Requirement For Having Certified Operators

In accordance with AEPEA, day to day operations of wastewater systems should be supervised by one or more persons who hold a valid certificate of qualification for the type of class of facility concerned. The Approval for each facility will state the required number of certified operators and their required level of certification.

Exempted from these requirement are:

1. "Privately owned developments" as defined in the regulations; and
2. Any other systems as determined by AEP.

10.1.4.3 Responsibility Of Operators

It is the responsibility of certified operators to know and understand the terms and conditions in the operating Approval for their facility. It is also their responsibility to understand the certification requirements for operators of their facilities as indicated by the Approval or by the Certification Guidelines.

It is necessary that the chief operator ensures current certification for operators as required by the Approval or by the Certification Guidelines. It is also important that each facility has a contingency plan so that certified operator requirements are met in cases of planned absences (eg., vacation), unplanned absences (eg., illness), or change of staff (eg., retirement).

Certified operators are also responsible to establish or understand contingency plans for each facility that ensure that the Approval requirements, with respect to certified operators, are met at all times. This is important during normal operation or in the cases of planned absences (eg. vacation), unplanned absences (eg. illness), or change of staff (eg. retirement).

10.1.4.4 Responsibility of Facility Owners

It is the legal responsibility of the owner of each facility to be aware of the requirements of the Approval and to ensure that the requirements are met. The Approval issued by AEP will designate the minimum number and level of certification of key operations personnel. It is important that facility owners develop an internal program so that substitute or replacement personnel are available when necessary.

TABLE 10.1(a)
CLASSIFICATION OF WASTEWATER TREATMENT PLANTS (WWT)

ITEM	POINTS
Size	
Maximum population equivalent (P.E.) served, peak day (1 pt/10,000 P.E. or part)	0 - 5
Design flow (avg. day) or peak month's flow (avg. day), whichever is larger (1 pt/5,000 m^3/day)	0 - 5
Effluent Discharge	
Receiving stream (sensitivity)*	0 - 6*
Land disposal - evaporation	2
Subsurface disposal	4
Variation in Raw Wastewater (slight to extreme)*	0 - 6*
Pretreatment	
Plant pumping of main flow	3
Screening, comminution	3
Grit removal	3
Chemical pre-treatment except chlorination, enzymes	4
Primary Treatment	
Primary clarifiers	5
Combined Sedimentation/digestion	5
Secondary Treatment	
Trickling filter w/sec. clarifiers or RBC	10
Activated sludge w/sec. clarifiers (including ext. aeration and oxidation ditches)	15
Stabilization ponds without aeration	5
Aerated lagoon	8
Advanced Waste Treatment	
Polishing pond	2
Chemical/physical - without secondary	15
Chemical/physical - following secondary	10
Biological or chemical/biological	12
Ion exchange	10
Reverse osmosis, electrodialysis	15
Chemical recovery, carbon regeneration.	4
Solids Handling	
Conditioning	2
Thickening	5
Anaerobic digestion	10
Aerobic digestion	6
Evaporative sludge drying	2
Mechanical dewatering	8
Solids reduction (incineration, wet oxidation)	12
Disinfection	
Chlorination or comparable	5
On-site generation	5
Laboratory Control by Plant Personnel	
Bacteriological (complexity)*	0 - 10*
Chemical/physical (complexity)*	0 - 10*

Note: Each unit process should have points assigned only once.

* See Table 10.1(b)

TABLE 10.1(b)
WASTEWATER TREATMENT PLANT CLASSIFICATION POINT GUIDE

ITEM	POINTS
Receiving Stream Sensitivity	0 - 6
The key factor is the degree of dilution provided under low flow conditions.	
Suggested point values are:	
No discharge	0
Secondary treatment is adequate.	1
Tertiary treatment is required	2
Stream conditions are very critical (run dry, for example)	3
Effluent used in a direct recycle and reuse system	6
Variation in Raw Wastewater Quality	
The key factor is the frequency and/or the extent of variation from normal or typical fluctuations; such deviation can be in terms of strength, toxicity, shock loads, etc.	
Suggested point values are:	
Variations do not exceed those normally or typically expected	0
Recurring deviations or excessive variations of 100 to 200 percent in strength and/or flow	2
Recurring deviations or excessive variations of more than 200 percent in strength and/or flow	4
Raw wastes subject to toxic waste discharges	6
Laboratory Control By Plant Personnel	
Bacteriological/biological (complexity)	0 - 10
The key concept is to credit bacti/bio lab work done on-site by plant personnel.	
Suggested point values are:	
Lab work done outside the plant	0
Membrane filter procedures	3
Use of fermentation tubes or any dilution methods; fecal coliform determination	5
Biological identification	7
Virus/parasite studies or similarly complex work conducted on-site	10
Chemical/physical (complexity)	0 - 10
The key concept is to credit chemical/physical lab work done on-site by plant personnel.	
Suggested point values are:	
Lab work done outside the plant	0
Push button or visual methods for simple test such as pH, settleable solids - up to	3
Additional procedures such as DO, COD, BOD, solids, gas analysis, titrations, volatile content - up to	5
More advanced determinations such as specific constituents: nutrients, total oils, phenols, etc.	7
Highly sophisticated instrumentation such as atomic absorption and gas chromatography	10

TABLE 10.2

FACILITY CLASSIFICATION SYSTEM

FACILITY	BASED UPON	I	II	III	IV
WWC*	Population Served	1500 or fewer	1501-15,000	15,001-50,000	50,001 or more
WWT	Range of Points (Table 10.1(a))	30 or fewer	31-55	56-75	76 or more

Notes: AEP may adjust the classification of a facility if the point system does not reflect the actual complexity of that facility.

WWC - Wastewater Collection
WWT - Wastewater Treatment

Wastewater pumping and transmission may be part of either wastewater treatment or wastewater collection but, alone, it is not considered to be either wastewater treatment or wastewater collection.

*Simple "in-line" treatment (such as odour control) is considered an integral part of the collection system.

10.1.4.5 Facilities Staffing Requirements: Certified Operators

For Class I facilities, there must be a certified Small Systems or Level I (or higher) operator in charge of the day to day operation of that facility. A back-up certified operator is **recommended**.

For Class II facilities, there must be a certified Level II (or higher) operator in charge of the day to day operation of that facility. It is recommended that an assistant operator with Level I or II certification be available.

For Class III facilities serving a population under 1,500, there must be a certified Level III (or higher) operator in charge of the day to day operation.

For Class III facilities serving a population of 1,500 - 10,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be at least one other operator certified at Level I or higher.

For Class III facilities serving a population of 10,000 - 50,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be at least one other operator certified at Level II or higher.

For Class III facilities serving a population over 50,000, there must be a certified Level III (or higher) operator in charge of the day to day operation. There must also be another operator certified at Level II or higher to act in the absence of the charge operator and at least one other operator certified at Level I or higher. There must be at least one certified operator (any level) for each shift when shift operation is required.

For Class IV facilities serving a population up to 200,000, there must be a Level IV operator in charge of the day to day operation. There must also be two Level III (or higher) operators to act in the absence of the Level IV operator. There must be at least one Level II or higher certified operator for each shift when shift operation is required.

For Class IV facilities serving a population over 200,000, there must be a certified Level IV operator in charge of the day to day operation. There must be at least one other certified Level IV operator to act in the absence of the charge operator. There must be a third operator who is certified at either Level III or IV and there must be at least one Level II (or higher) certified operator for each shift when shift operation is required.

10.2 System Monitoring

10.2.1 General

AEP considers monitoring to fall into one of the following categories:

1. Operational monitoring
2. Treatment Performance and Compliance monitoring
3. Issue oriented and Follow-up monitoring.

Types of monitoring are discussed in detail in the next few sections.

10.2.1.1 Sampling Procedures and Analytical Methods

1. The usefulness of any monitoring program depends to a large extent on the sampling procedures used. It is important to ensure that the sample collected is truly representative of the wastewater stream.

2. Based on the type of analysis, AEP would require two types of samples to be collected, "composite" or "grab."

   "Composite sample" means a sample consisting of not less that twenty-four discrete portions of equal volume collected as follows:

   i) at time intervals directly proportional to the flow rate of the liquid being sampled during each time interval, with a minimum of one discrete sample collected every hour over a period of twenty-four hours, or

   ii) sequentially at regular time intervals over a period of twenty-four hours.

   "Grab sample" means an individual sample collected in less than 15 minutes and which is representative of the wastewater being sampled.

3. The owner should ensure that:

   i) Collection and preservation of samples and all analytical procedures are:

      - in accordance with the latest edition of "Standard Methods for the Examination of Water and Wastewater," as published by the American Public Health Association, American Water Works Association, and the Water Pollution Control Federation; or

- by a method outlined in the most recent edition of the "Methods Manual for Chemical Analysis of Water and Wastes" or "Methods Manual for Chemical Analysis of Trace Organics and Pesticides in Environmental Samples," published by Alberta Environmental Protection; or

- by an alternative method approved by AEP; or

ii) Collection, preservation and the analysis of samples are performed by a laboratory approved by AEP.

10.2.1.2 Approval of Analytical Procedures

Owners should ensure that laboratories obtain approval from AEP for the use of any analytical procedures not included in the "Standard methods."

The laboratory would be required to follow a protocol established by AEP for the approval of analytical procedures not included in the "Standard Methods."

10.2.2 Operational Monitoring

The extent and the complexity of operational monitoring is dependent on the size and the type of the facility. For instance, there are a number of operating variables which are vitally important to the proper functioning of an activated sludge system. Some of these are under the Operator's control, and some are not. Table 10.3 lists some of the significant parameters that should be monitored to ensure proper operation of an activated sludge system. Operational monitoring requirements are established both for specific process control purposes, and to ensure that a facility receives good operational attention on a regular basis.

Table 10.4 outlines the operational monitoring requirements for wastewater stabilization ponds.

10.2.2.1 Activated Sludge Systems

TABLE 10.3

OPERATIONAL MONITORING

Parameters	Point of Measurement	Requirement	Type of Sample	Monitoring Frequency
Flow (raw)	Headworks	Not to exceed design capacity	N/A	Daily from totalizer
Flow (treated)	Prior to Outfall	N/A	N/A	Daily from totalizer
Peakflow (raw)	Headworks	N/A	N/A	Daily from strip - chart or PLC
TSS (raw)	Headworks	N/A	Composite	Daily
TSS (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
TSS (RAS)	RAS line	Varies; required to control solids in aeration tank	Grab/Composite	Daily
TSS (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
MLSS	Aeration tank	800-2000 mg/L (without nitrification) 2000-4000 mg/L (with nitrification)	Grab Grab	Daily Daily
CBOD (raw)	Headworks	N/A	Composite	Daily
COD (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
CBOD (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
Sludge Volume Index	Aeration tank/calculated	< 150 mL/g	Grab	Daily
Sludge Age	Calculated	3-10 days (without nitrification) 10-15 days (with nitrification)	N/A N/A	Daily Daily
F:M ratio	Calculated	0.05 - 0.5	N/A	Twice Weekly
Dissolved Oxygen	Aeration tank	2 mg/L	N/A	Continuous

10.2.2.2 Wastewater Lagoons

TABLE 10.4

OPERATIONAL MONITORING

Requirements	Monitoring Frequency
Actual dates and duration of discharge	Annually for every discharge
Volume of discharge	Annually for every discharge
Monitoring of each groundwater observation well for: water level, TKN NH_3-N, NO_3-N, NO_2-N, TDS, COD, and any other parameter as determined by AEP.	For new lagoons, four times in each quarter of the first year of operation. The first analysis from each well prior to putting the new lagoon into operation. The following three analyses approximately three months apart to cover all seasons of the year.
Monitoring of each groundwater observation well for water level during the discharge period.	One set of readings immediately before discharge, one set immediately after discharge, and one set approximately one month after the end of the discharge period.

10.2.3 Treatment Performance and Compliance Monitoring

Treatment performance and compliance monitoring will be dependent on a number of factors, including:

- type of treatment (mechanical secondary treatment plants, mechanical tertiary treatment plants, aerated lagoons, wastewater stabilization ponds)
- type of discharge (continuous, intermittent)
- type of receiving body (water or land)

The wastewater and storm drainage regulation and the wastewater and Storm Drainage (ministerial) Regulation (119/93 and 120/93) require physical, microbiological, radiological or chemical analyses of wastewater and storm drainage samples for those parameters specified by AEP. This section outlines the specific parameters that have to be monitored, including the sample location and the monitoring frequency, for different types of treatment method, discharge and receiving body.

10.2.3.1 Secondary Treatment - Mechanical (for current population < 20,000)

1. Continuous Discharge to a body of water

TABLE 10.5

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of daily samples shall not exceed 25 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 25 mg/L	Composite	Daily

2. Intermittent Discharge to a Body of Water

TABLE 10.6

TREATMENT PERFORMANCE AT COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of three times per week; samples shall not exceed 25 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period **should** not exceed three weeks.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 25 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
TSS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. Continuous and/or intermittent discharge to land (Effluent Irrigation)

TABLE 10.7

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of **a** major application event
TSS	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of **a** major application event
EC	Prior to irrigation	< 2.5 mmhos/cm	Grab/composite*	Twice annually, prior to and on completion of **a** major application event
SAR	Prior to irrigation	< 9	Grab/composite*	Twice annually, prior to and on completion of **a** major application event
pH	Prior to irrigation	6.5 - 9.5	Grab/composite*	Twice annually, prior to and on completion of **a** major application event
Total Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 1000/100 mL	Grab	Daily**/weekly during the irrigation season
Fecal Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 200/100 mL	Grab	Daily**/weekly during the irrigation season

***** Grab sample if storage provided; composite sample if storage not provided

** Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

10.2.3.2 Tertiary Treatment - Mechanical (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 10.8

TREATMENT PERFORMANCE AND COMPLIANCE

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TP	Prior to outfall	Monthly average of daily samples shall not exceed 1 mg/L	Composite	Daily
NH_3-N	Prior to outfall	Assessed on a site specific basis	Composite	Assessed on a site specific basis
Total Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 1000/100 mL	Grab	*Daily
Fecal Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 200/100 mL	Grab	*Daily

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

2. Intermittent Discharge to a Body of Water

TABLE 10.9

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of three times per week samples shall not exceed 20 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period shall not exceed three weeks, unless local conditions preclude this rate of discharge.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 20 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
TSS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of five times per week samples shall not exceed 1 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays

Table 10.9 - Continued

Parameter	**Point of Measurement**	**Requirement**	**Type of Sample**	**Minimum Monitoring Frequency**
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site specific basis.
Total Coliform	Prior to storage cell	Geometric mean of three times per week; samples in a calendar month shall not exceed 1000/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of three times per week samples in a calendar month shall not exceed 200/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality is consistent and the probability of variance is minimal.

3. **Continuous and/or intermittent discharge to land (Effluent Irrigation)**
Tertiary (mechanical) treatment performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. Please refer to table 10.7.

10.2.3.3 Aerated Lagoons (for current population < 20,000)

1. Continuous discharge to a body of water

TABLE 10.10
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of weekly samples shall not exceed 25 mg/L	Grab	weekly

2. Intermittent discharges to a body of water

TABLE 10.11
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 25 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. Continuous and/or intermittent discharge to land (Effluent Irrigation)

Aerated lagoon performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. please refer to table 10.7.

10.2.3.4 Aerated Lagoons (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 10.12

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	weekly
TP	Prior to outfall	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	weekly
NH_3-N	Prior to outfall	Assessed on site specific basis	Grab	Assessed on site specific basis.
Total Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 mL	Grab	weekly
Fecal Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 mL	Grab	weekly

2. Intermittent discharge to a body of water

TABLE 10.13

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	Weekly during the period of discharge to storage cell.
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site specific basis
Total Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 ml	Grab	Weekly during the period of discharge to storage cell
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 ml	Grab	Weekly during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local

conditions preclude this rate of discharge.

10.2.3.5 Wastewater Lagoons

TABLE 10.14

TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Type	Minimum Requirements
Wastewater stabilization pond built to the specified design configuration as per section 5.2.1.2.	Effluent quality standard not specified Discharge from the storage cell once a year between late spring and fall. Early spring discharged allowed under exceptional circumstances Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

10.2.4 Issue Oriented and Follow-up Monitoring

1. Follow-up action by the owner may be required when the system does not meet or produce effluent to meet the standards stipulated in Section 3.0: Performance Standards - Wastewater. Issue oriented monitoring and follow-up actions for various incidents are outlined in Table 10.15.

2. When a violation of the prescribed effluent standard occurs, the owner should:

   (i) Notify AEP in accordance with Section 10.4;
   (ii) Determine the cause of the problem; and
   (iii) Take action as directed by AEP.

10.2.4.1 Activated Sludge System

TABLE 10.15

ISSUE ORIENTED AND FOLLOW-UP MONITORING

Incident	Parameter	Point of Measurement	Type of Sample	Monitoring Frequency	Follow-Up
Plant by-pass	flow	By-pass line	N/A	Each incident	-
	CBOD		Grab/composite		
	TSS		Grab/composite		
	Coliform (total)		Grab		
	Coliform (fecal)		Grab		
	NH_3		Grab/composite		
	Total.P		Grab/composite		
Sludge bulking	CB0D (primary)	Downstream of primary clarifier	Grab/composite	For the duration of event	Adjust sludge age, adjust D.O levels, Adjust WAS rates, chlorination of RAS
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	pH (primary))	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	D.O.	aeration tank	N/A		
	NO_3-N	RAS line	Grab		
Rising Sludge/ Denitrification	CBOD (primary)	Downstream of primary clarifier	Grab/composite	For the duration of the event	Increase F/M ratio, Adjust RAS, Adjust D.O. levels, Step-feed influent
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	NO_3-N	RAS line	Grab		
Primary Sludge Septicity	Flow				
	Retention time				
	Density				
	CBOD (raw)				
	TSS (raw)				

Primary sludge line	N/A	For the duration of the event	Increase sludge wasting, monitor retention time

10.3 Record Keeping

All records should bear the signature of the operator in responsible charge of the wastewater system or his or her representative. Owners shall keep these records available for inspection by AEP and shall send the records to AEP if requested.

The owners shall keep the following records and effluent quality analyses:

1. All daily records for treatment performance and compliance monitoring for five years.

2. All daily records for operational monitoring for three years.

3. Actual laboratory reports may be kept or data may be transferred to tabular summaries, provided the following information is included:

   i) The date, place and time of sampling, and the name of the person collecting the sample;

   ii) Identification of the sample type (compliance sample, operational sample, special purpose sample);

   iii) Date of analysis;

   iv) Laboratory and person responsible for performing analysis;

   v) The analytical method used; and

   vi) The results of the analysis.

4. Records of action taken by the system to correct violations of effluent standards or operating approval requirements.

5. Copies of project reports, construction documents and related drawings, inspection reports and approvals should be kept for the life of the facility.

10.4 Reporting

1. Reporting requirements shall be in accordance with the operating approval for the facility, issued by AEP.

2. The owner shall report to AEP within one week:

   i) The violation of prescribed effluent standards for that facility; and

   ii) The failure to comply with the treatment performance and compliance monitoring requirements.

3. Immediate notification by telephone to 1-800-222-6514 shall be made, followed by a written report to AEP within one week and remedial action carried out as per EPEA Division 1, Sections 99, 100 and 101 in the event of:

   - any discharge of raw or partially treated wastewater
   - any unauthorized discharge or accidental spill of raw or partially treated wastewater
   - any overflow from the wastewater collection or treatment system

4. At least one week prior to draining of wastewater stabilization ponds, the owner shall notify AEP in writing of the proposed discharging schedule.

5. The owner should compile and submit an annual report on or before February 28 of the following year on which the information was collected. The report should include the following:

   i) A monthly summary of all operational and compliance monitoring for that particular facility, as identified by AEP;

   ii) A summary of approval contraventions and remedial measures taken; and

   iii) A summary of any permanent upgrading works undertaken during the year.

APPENDIX A

CT VALUES FOR INACTIVATION OF <u>GIARDIA</u> CYSTS BY FREE CHLORINE

Adopted from <u>Optimizing Water Treatment Plant Performance Using the Composite Correction Program (1991)</u>, prepared by Process Applications Inc., Fort Collins, Colo., for the US Environmental Protection Agency, Office of Drinking Water, Cincinnati, Ohio

TABLE A-1

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 0.5^{o}C OR LOWER

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	23	46	69	91	114	137	27	54	82	109	136	163	33	65	98	130	163	195	40	79	119	158	198	237
0.6	24	47	71	94	118	141	28	56	84	112	140	168	33	67	100	133	167	200	40	80	120	159	199	239
0.8	24	48	73	97	121	145	29	57	86	115	143	172	34	68	103	137	171	205	41	82	123	164	205	246
1	25	49	74	99	123	148	29	59	88	117	147	176	35	70	105	140	175	210	42	84	127	169	211	253
1.2	25	51	76	101	127	152	30	60	90	120	150	180	36	72	108	143	179	215	43	86	130	173	216	259
1.4	26	52	78	103	129	155	31	61	92	123	153	184	37	74	111	147	184	221	44	89	133	177	222	266
1.6	26	52	79	105	131	157	32	63	95	126	158	189	38	75	113	151	188	226	46	91	137	182	228	273
1.8	27	54	81	108	135	162	32	64	97	129	161	193	39	77	116	154	193	231	47	93	140	186	233	279
2	28	55	83	110	138	165	33	66	99	131	164	197	39	79	118	157	197	236	48	95	143	191	238	286
2.2	28	56	85	113	141	169	34	67	101	134	168	201	40	81	121	161	202	242	50	99	149	198	248	297
2.4	29	57	86	115	143	172	34	68	103	137	171	205	41	82	124	165	206	247	50	99	149	199	248	298
2.6	29	58	88	117	146	175	35	70	105	139	174	209	42	84	126	168	210	252	51	101	152	203	253	304
2.8	30	59	89	119	148	178	36	71	107	142	178	213	43	86	129	171	214	257	52	103	155	207	258	310
3	30	60	91	121	151	181	36	72	109	145	181	217	44	87	131	174	218	261	53	105	158	211	263	316

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	46	92	139	185	231	277	55	110	165	219	274	329	65	130	195	260	325	390
0.6	48	95	143	191	238	286	57	114	171	228	285	342	68	136	204	271	339	407
0.8	49	98	148	197	246	295	59	118	177	236	295	354	70	141	211	281	352	422
1	51	101	152	203	253	304	61	122	183	243	304	365	73	146	219	291	364	437
1.2	52	104	157	209	261	313	63	125	188	251	313	376	75	150	226	301	376	451
1.4	54	107	161	214	268	321	65	129	194	258	323	387	77	155	232	309	387	464
1.6	55	110	165	219	274	329	66	132	199	265	331	397	80	159	239	318	398	477
1.8	56	113	169	225	282	338	68	136	204	271	339	407	82	163	245	326	408	489
2	58	115	173	231	288	346	70	139	209	278	348	417	83	167	250	333	417	500
2.2	59	118	177	235	294	353	71	142	213	284	355	426	85	170	256	341	426	511
2.4	60	120	181	241	301	361	73	145	218	290	363	435	87	174	261	348	435	522
2.6	61	123	184	245	307	368	74	148	222	296	370	444	89	178	267	355	444	533
2.8	63	125	188	250	313	375	75	151	226	301	377	452	91	181	272	362	453	543
3	64	127	191	255	318	382	77	153	230	307	383	460	92	184	276	368	460	552

TABLE A-2

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT $5^{o}C$

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5 `	1.0	1.5	2.0	2.5	3.0
< = 0.4	16	32	49	65	81	97	20	39	59	78	98	117	23	46	70	93	116	139	28	55	83	111	138	166
0.6	17	33	50	67	83	100	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	114	143	171
0.8	17	34	52	69	86	103	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175
1	18	35	53	70	88	105	21	42	63	83	104	125	25	50	75	99	124	149	30	60	90	119	149	179
1.2	18	36	54	71	89	107	21	42	64	85	106	127	25	51	76	101	127	152	31	61	92	122	153	183
1.4	18	36	55	73	91	109	22	43	65	87	108	130	26	52	78	103	129	155	31	62	94	125	156	187
1.6	19	37	56	74	93	111	22	44	66	88	110	132	26	53	79	105	132	158	32	64	96	128	160	192
1.8	19	38	57	76	95	114	23	45	68	90	113	135	27	54	81	108	135	162	33	65	98	131	163	196
2	19	39	58	77	97	116	23	46	69	92	115	138	28	55	83	110	138	165	33	67	100	133	167	200
2.2	20	39	59	79	98	118	23	47	70	93	117	140	28	56	85	113	141	169	34	68	102	136	170	204
2.4	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	115	143	172	35	70	105	139	174	209
2.6	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175	36	71	107	142	178	213
2.8	21	41	62	83	103	124	25	49	74	99	123	148	30	59	89	119	148	178	36	72	109	145	181	217
3	21	42	63	84	105	126	25	50	76	101	126	151	30	61	91	121	152	182	37	74	111	147	184	221

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	33	66	99	132	165	198	39	79	118	157	197	236	47	93	140	186	233	279
0.6	34	68	102	136	170	204	41	81	122	163	203	244	49	97	146	194	243	291
0.8	35	70	105	140	175	210	42	84	126	168	210	252	50	100	151	201	251	301
1	36	72	108	144	180	216	43	87	130	173	217	260	52	104	156	208	260	312
1.2	37	74	111	147	184	221	45	89	134	178	223	267	53	107	160	213	267	320
1.4	38	76	114	151	189	227	46	91	137	183	228	274	55	110	165	219	274	329
1.6	39	77	116	155	193	232	47	94	141	187	234	281	56	112	169	225	281	337
1.8	40	79	119	159	198	238	48	96	144	191	239	287	58	115	173	230	288	345
2	41	81	122	162	203	243	49	98	147	196	245	294	59	118	177	235	294	353
2.2	41	83	124	165	207	248	50	100	150	200	250	300	60	120	181	241	301	361
2.4	42	84	127	169	211	253	51	102	153	204	255	306	61	123	184	245	307	368
2.6	43	86	129	172	215	258	52	104	156	208	260	312	63	125	188	250	313	375
2.8	44	88	132	175	219	263	53	106	159	212	265	318	64	127	191	255	318	382
3	45	89	134	179	223	268	54	108	162	216	270	324	65	130	195	259	324	389

TABLE A-3

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 10^{o}C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	24	37	49	61	73	15	29	44	59	73	88	17	35	52	69	87	104	21	42	63	83	104	125
0.6	13	25	38	50	63	75	15	30	45	60	75	90	18	36	54	71	89	107	21	43	64	85	107	128
0.8	13	26	39	52	65	78	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131
1	13	26	40	53	66	79	16	31	47	63	78	94	19	37	56	75	93	112	22	45	67	89	112	134
1.2	13	27	40	53	67	80	16	32	48	63	79	95	19	38	57	76	95	114	23	46	69	91	114	137
1.4	14	27	41	55	68	82	16	33	49	65	82	98	19	39	58	77	97	116	23	47	70	93	117	140
1.6	14	28	42	55	69	83	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	96	120	144
1.8	14	29	43	57	72	86	17	34	51	67	84	101	20	41	61	81	102	122	25	49	74	98	123	147
2	15	29	44	58	73	87	17	35	52	69	87	104	21	41	62	83	103	124	25	50	75	100	125	150
2.2	15	30	45	59	74	89	18	35	53	70	88	105	21	42	64	85	106	127	26	51	77	102	128	153
2.4	15	30	45	60	75	90	18	36	54	71	89	107	22	43	65	86	108	129	26	52	79	105	131	157
2.6	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131	27	53	80	107	133	160
2.8	16	31	47	62	78	93	19	37	56	74	93	111	22	45	67	89	112	134	27	54	82	109	136	163
3	16	32	48	63	79	95	19	38	57	75	94	113	23	46	69	91	114	137	28	55	83	111	138	166

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	25	50	75	99	124	149	30	59	89	118	148	177	35	70	105	139	174	209
0.6	26	51	77	102	128	153	31	61	92	122	153	183	36	73	109	145	182	218
0.8	26	53	79	105	132	158	32	63	95	126	158	189	38	75	113	151	188	226
1	27	54	81	108	135	162	33	65	98	130	163	195	39	78	117	156	195	234
1.2	28	55	83	111	138	166	33	67	100	133	167	200	40	80	120	160	200	240
1.4	28	57	85	113	142	170	34	69	103	137	172	206	41	82	124	165	206	247
1.6	29	58	87	116	145	174	35	70	106	141	176	211	42	84	127	169	211	253
1.8	30	60	90	119	149	179	36	72	108	143	179	215	43	86	130	173	216	259
2	30	61	91	121	152	182	37	74	111	147	184	221	44	88	133	177	221	265
2.2	31	62	93	124	155	186	38	75	113	150	188	225	45	90	136	181	226	271
2.4	32	63	95	127	158	190	38	77	115	153	192	230	46	92	138	184	230	276
2.6	32	65	97	129	162	194	39	78	117	156	195	234	47	94	141	187	234	281
2.8	33	66	99	131	164	197	40	80	120	159	199	239	48	96	144	191	239	287
3	34	67	101	134	168	201	41	81	122	162	203	243	49	97	146	195	243	292

TABLE A-4

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 15^{o}C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	8	16	25	33	41	49	10	20	30	39	49	59	12	23	35	47	58	70	14	28	42	55	69	83
0.6	8	17	25	33	42	50	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86
0.8	9	17	26	35	43	52	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88
1	9	18	27	35	44	53	11	21	32	42	53	63	13	25	38	50	63	75	15	30	45	60	75	90
1.2	9	18	27	36	45	54	11	21	32	43	53	64	13	25	38	51	63	76	15	31	46	61	77	92
1.4	9	18	28	37	46	55	11	22	33	43	54	65	13	26	39	52	65	78	16	31	47	63	78	94
1.6	9	19	28	37	47	56	11	22	33	44	55	66	13	26	40	53	66	79	16	32	48	64	80	96
1.8	10	19	29	38	48	57	11	23	34	45	57	68	14	27	41	54	68	81	16	33	49	65	82	98
2	10	19	29	39	48	58	12	23	35	46	58	69	14	28	42	55	69	83	17	33	50	67	83	100
2.2	10	20	30	39	49	59	12	23	35	47	58	70	14	28	43	57	71	85	17	34	51	68	85	102
2.4	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86	18	35	53	70	88	105
2.6	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88	18	36	54	71	89	107
2.8	10	21	31	41	52	62	12	25	37	49	62	74	15	30	45	59	74	89	18	36	55	73	91	109
3	11	21	32	42	53	63	13	25	38	51	63	76	15	30	46	61	76	91	19	37	56	74	93	111

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	17	33	50	66	83	99	20	39	59	79	98	118	23	47	70	93	117	140
0.6	17	34	51	68	85	102	20	41	61	81	102	122	24	49	73	97	122	146
0.8	18	35	53	70	88	105	21	42	63	84	105	126	25	50	76	101	126	151
1	18	36	54	72	90	108	22	43	65	87	108	130	26	52	78	104	130	156
1.2	19	37	56	74	93	111	22	45	67	89	112	134	27	53	80	107	133	160
1.4	19	38	57	76	95	114	23	46	69	91	114	137	28	55	83	110	138	165
1.6	19	39	58	77	97	116	24	47	71	94	118	141	28	56	85	113	141	169
1.8	20	40	60	79	99	119	24	48	72	96	120	144	29	58	87	115	144	173
2	20	41	61	81	102	122	25	49	74	98	123	147	30	59	89	118	148	177
2.2	21	41	62	83	103	124	25	50	75	100	125	150	30	60	91	121	151	181
2.4	21	42	64	85	106	127	26	51	77	102	128	153	31	61	92	123	153	184
2.6	22	43	65	86	108	129	26	52	78	104	130	156	31	63	94	125	157	188
2.8	22	44	66	88	110	132	27	53	80	106	133	159	32	64	96	127	159	191
3	22	45	67	89	112	134	27	54	81	108	135	162	33	65	98	130	163	195

TABLE A-5

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 20^{o}C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations						pH = 6.5 Log Inactivations						pH = 7.0 Log Inactivations						pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	6	12	18	24	30	36	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62
0.6	6	13	19	25	32	38	8	15	23	30	38	45	9	18	27	36	45	54	11	21	32	43	53	64
0.8	7	13	20	26	33	39	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66
1	7	13	20	26	33	39	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67
1.2	7	13	20	27	33	40	8	16	24	32	40	48	10	19	29	38	48	57	12	23	35	46	58	69
1.4	7	14	21	27	34	41	8	16	25	33	41	49	10	19	29	39	48	58	12	23	35	47	58	70
1.6	7	14	21	28	35	42	8	17	25	33	42	50	10	20	30	39	49	59	12	24	36	48	60	72
1.8	7	14	22	29	36	43	9	17	26	34	43	51	10	20	31	41	51	61	12	25	37	49	62	74
2	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62	13	25	38	50	63	75
2.2	7	15	22	29	37	44	9	18	27	35	44	53	11	21	32	42	53	63	13	26	39	51	64	77
2.4	8	15	23	30	38	45	9	18	27	36	45	54	11	22	33	43	54	65	13	26	39	52	65	78
2.6	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66	13	27	40	53	67	80
2.8	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67	14	27	41	54	68	81
3	8	16	24	31	39	47	10	19	29	38	48	57	11	23	34	45	57	68	14	28	42	55	69	83

Chlorine Concentrations (mg/L)	pH = 8.0 Log Inactivations						pH = 8.5 Log Inactivations						pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	25	37	49	62	74	15	30	45	59	74	89	18	35	53	70	88	105
0.6	13	26	39	51	64	77	15	31	46	61	77	92	18	36	55	73	91	109
0.8	13	26	40	53	66	79	16	32	48	63	79	95	19	38	57	75	94	113
1	14	27	41	54	68	81	16	33	49	65	82	98	20	39	59	78	98	117
1.2	14	28	42	55	69	83	17	33	50	67	83	100	20	40	60	80	100	120
1.4	14	28	43	57	71	85	17	34	52	69	86	103	21	41	62	82	103	123
1.6	15	29	44	58	73	87	18	35	53	70	88	105	21	42	63	84	105	126
1.8	15	30	45	59	74	89	18	36	54	72	90	108	22	43	65	86	108	129
2	15	30	46	61	76	91	18	37	55	73	92	110	22	44	66	88	110	132
2.2	16	31	47	62	78	93	19	38	57	75	94	113	23	45	68	90	113	135
2.4	16	32	48	63	79	95	19	38	58	77	96	115	23	46	69	92	115	138
2.6	16	32	49	65	81	97	20	39	59	78	98	117	24	47	71	94	118	141
2.8	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	95	119	143
3	17	34	51	67	84	101	20	41	61	81	102	122	24	49	73	97	122	146

TABLE A-6

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 25^{O}C

Chlorine Concentrations (mg/L)	pH < = 6 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	4	8	12	16	20	24
0.6	4	8	13	17	21	25
0.8	4	9	13	17	22	26
1	4	9	13	17	22	26
1.2	5	9	14	18	23	27
1.4	5	9	14	18	23	27
1.6	5	9	14	19	23	28
1.8	5	10	15	19	24	29
2	5	10	15	19	24	29
2.2	5	10	15	20	25	30
2.4	5	10	15	20	25	30
2.6	5	10	16	21	26	31
2.8	5	10	16	21	26	31
3	5	11	16	21	27	32

Chlorine Concentrations (mg/L)	pH = 6.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	5	10	15	19	24	29
0.6	5	10	15	20	25	30
0.8	5	10	16	21	26	31
1	5	10	16	21	26	31
1.2	5	11	16	21	27	32
1.4	6	11	17	22	28	33
1.6	6	11	17	22	28	33
1.8	6	11	17	23	28	34
2	6	12	18	23	29	35
2.2	6	12	18	23	29	35
2.4	6	12	18	24	30	36
2.6	6	12	19	25	31	37
2.8	6	12	19	25	31	37
3	6	13	19	25	32	38

Chlorine Concentrations (mg/L)	pH = 7.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	6	12	18	23	29	35
0.6	6	12	18	24	30	36
0.8	6	12	19	25	31	37
1	6	12	19	25	31	37
1.2	6	13	19	25	32	38
1.4	7	13	20	26	33	39
1.6	7	13	20	27	33	40
1.8	7	14	21	27	34	41
2	7	14	21	27	34	41
2.2	7	14	21	28	35	42
2.4	7	14	22	29	36	43
2.6	7	15	22	29	37	44
2.8	8	15	23	30	38	45
3	8	15	23	31	38	46

Chlorine Concentrations (mg/L)	pH = 7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	7	14	21	28	35	42
0.6	7	14	22	29	36	43
0.8	7	15	22	29	37	44
1	8	15	23	30	38	45
1.2	8	15	23	31	38	46
1.4	8	16	24	31	39	47
1.6	8	16	24	32	40	48
1.8	8	16	25	33	41	49
2	8	17	25	33	42	50
2.2	9	17	26	34	43	51
2.4	9	17	26	35	43	52
2.6	9	18	27	35	44	53
2.8	9	18	27	36	45	54
3	9	18	28	37	46	55

Chlorine Concentrations (mg/L	pH = 8.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	8	17	25	33	42	50
0.6	9	17	26	34	43	51
0.8	9	18	27	35	44	53
1	9	18	27	36	45	54
1.2	9	18	28	37	46	55
1.4	10	19	29	38	48	57
1.6	10	19	29	39	48	58
1.8	10	20	30	40	50	60
2	10	20	31	41	51	61
2.2	10	21	31	41	52	62
2.4	11	21	32	42	53	63
2.6	11	22	33	43	54	65
2.8	11	22	33	44	55	66
3	11	22	34	45	56	67

Chlorine Concentrations (mg/L	pH = 8.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	10	20	30	39	49	59
0.6	10	20	31	41	51	61
0.8	11	21	32	42	53	63
1	11	22	33	43	54	65
1.2	11	22	34	45	56	67
1.4	12	23	35	46	58	69
1.6	12	23	35	47	58	70
1.8	12	24	36	48	60	72
2	12	25	37	49	62	74
2.2	13	25	38	50	63	75
2.4	13	26	39	51	64	77
2.6	13	26	39	52	65	78
2.8	13	27	40	53	67	80
3	14	27	41	54	68	81

Chlorine Concentrations (mg/L	pH < = 9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0
< = 0.4	12	23	35	47	58	70
0.6	12	24	37	49	61	73
0.8	13	25	38	50	63	75
1	13	26	39	52	65	78
1.2	13	27	40	53	67	80
1.4	14	27	41	55	68	82
1.6	14	28	42	56	70	84
1.8	14	29	43	57	72	86
2	15	29	44	59	73	88
2.2	15	30	45	60	75	90
2.4	15	31	46	61	77	92
2.6	16	31	47	63	78	94
2.8	16	32	48	64	80	96
3	16	32	49	65	81	97

APPENDIX B

CT VALUES FOR INACTIVATION OF <u>GIARDIA</u> CYSTS AND VIRUSES BY VARIOUS DISINFECTANTS

Adopted from <u>Strawman Regulations for Ground Water Disinfection</u>, US Environmental Protection Agency, Office of Drinking Water, Washington, D.C. (June 1990)

TABLE B-1

CT VALUES FOR INACTIVATION OF VIRUSES BY FREE CHLORINE*†

	Log Inactivation					
	2.0		3.0		4.0	
	pH		pH		pH	
Temperature (^{o}C)	**6-9**	**10**	**6-9**	**10**	**6-9**	**10**
0.5	6	45	9	66	12	90
5	4	30	6	44	8	60
10	3	22	4	33	6	45
15	2	15	3	22	4	30
20	1	11	2	16	3	22
25	1	7	1	11	2	15

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6, 7, 8, 9 and 10 and temperature = 5^{o}C. CT values include a safety factor of 3.

†CT values adjusted to other temperatures by doubling CT for each 10^{o}C drop in temperature.

TABLE B-2

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY CHLORINE DIOXIDE†*

	Temperature (^{o}C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
0.5 log	10	4.3	4	3.2	2.5	2
1 log	21	8.7	7.7	6.3	5	3.7
1.5 log	32	13	12	10	7.5	5.5
2 log	42	17	15	13	10	7.3
2.5 log	52	22	19	16	13	9
3 log	63	26	23	19	15	11

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5^{o}C.
CT values include a safety factor of 2.

†CT values adjusted to other temperatures by doubling CT for each 10^{o}C drop in temperature.

TABLE B-3

CT VALUES FOR INACTIVATION OF VIRUSES BY CHLORINE DIOXIDE PH 6-9*†

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
2 log	8.4	5.6	4.2	2.8	2.1	1.4
3 log	25.6	17.1	12.8	8.6	6.4	4.3
4 log	50.1	33.4	25.1	16.7	12.5	8.4

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH - 6.0 and temperature = 5°C.
CT values include a safety factor of 2.

†CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-4

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY OZONE*†

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
0.5 log	0.48	0.32	0.23	0.16	0.1	0.08
1 log	0.97	0.63	0.48	0.32	0.2	0.16
1.5 log	1.5	0.95	0.72	0.48	0.36	0.24
2 log	1.9	1.3	0.95	0.63	0.48	0.32
2.5 log	2.4	1.6	1.2	0.79	0.6	0.4
3 log	2.9	1.9	1.43	0.95	0.72	0.48

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH - 6.0 and temperature = 5°C.
CT values include a safety factor of 2.

†CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-5

CT VALUES FOR INACTIVATION OF VIRUSES BY OZONE*†

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
2 log	0.9	0.6	0.5	0.3	0.25	0.15
3 log	1.4	0.9	0.8	0.5	0.4	0.25
4 log	1.8	1.2	1.0	0.6	0.5	0.3

*Data adapted from Roy (1982) for inactivation of poliovirus for pH = 7.2 and temperature = 5°C. CT values include a safety factor of 3.

†CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-6

CT VALUES FOR INACTIVATION OF <u>GIARDIA</u> CYSTS BY CHLORAMINE pH 6-9*†

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
0.5 log	635	365	310	250	185	125
1 log	1270	735	615	500	370	250
1.5 log	1900	1100	930	750	550	375
2 log	2535	1470	1230	1000	735	500
2.5 log	3170	1830	1540	1250	915	625
3 log	3800	2200	1850	1500	1100	750

*Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6.0 and temperature = 5°C.
CT values include a safety factor of 2.

†CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE B-7

CT VALUES FOR INACTIVATION OF VIRUSES BY CHLORAMINE*†

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
2 log	1243	857	643	428	321	214
3 log	2063	1423	1067	712	534	356
4 log	2883	1988	1491	994	746	497

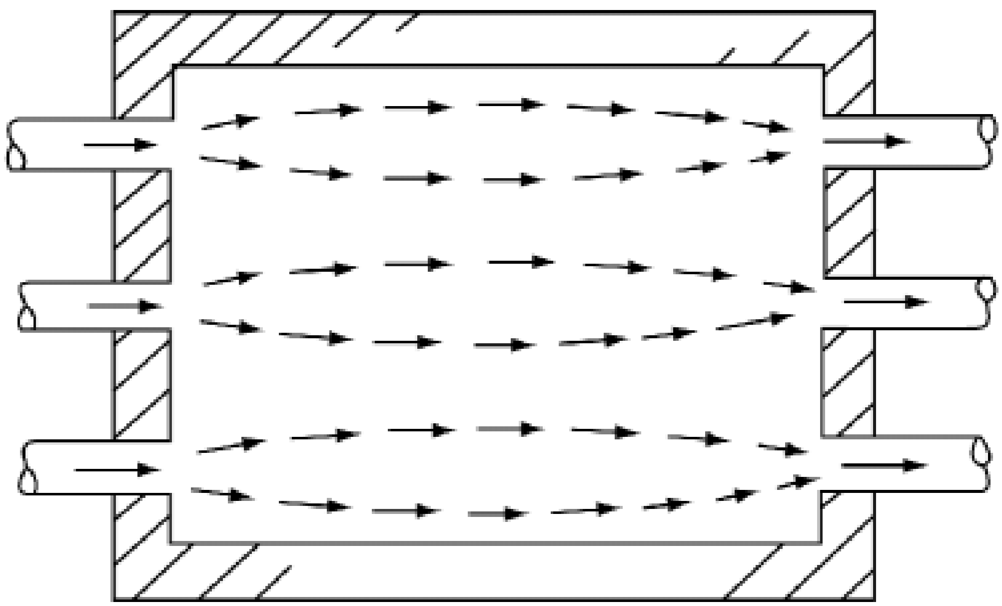
*Data adapted from Roy (1982) for inactivation of poliovirus for pH - 7.2 and temperature = 5°C. CT values include a safety factor of 3.

†CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

APPENDIX C

TYPICAL BAFFLING CONDITIONS

Poor Baffling Conditions - Rectangular Contact Basin



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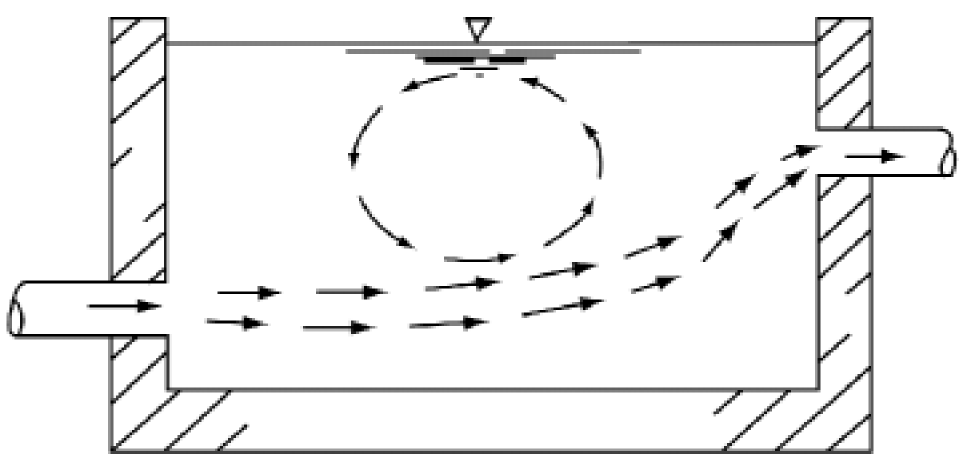
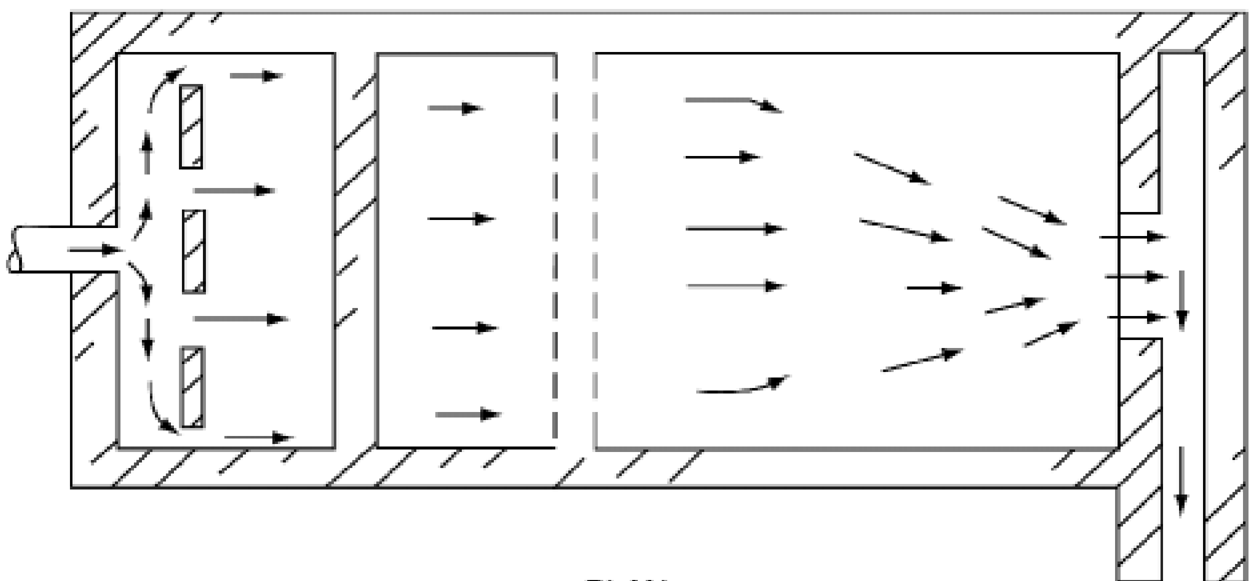


SECTION

Average Baffling Conditions - Rectangular Contact Basin



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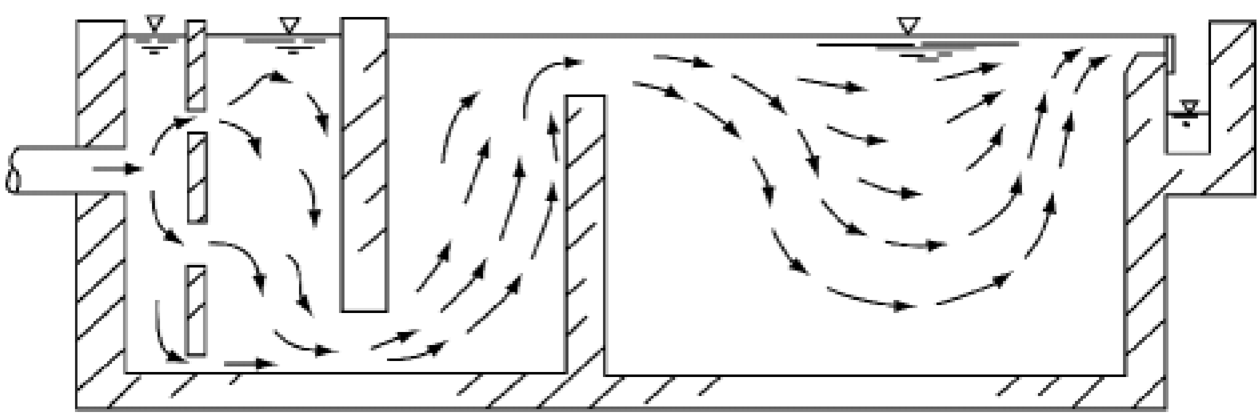

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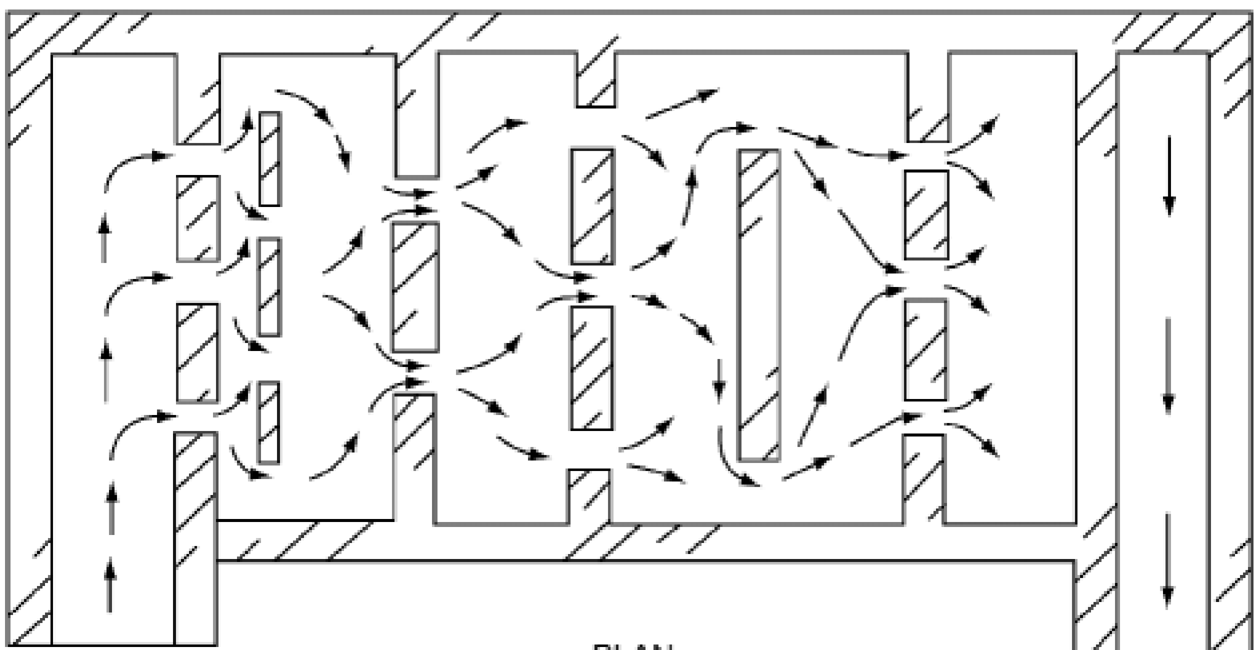
Superior Baffling Conditions - Rectangular Contact Basin



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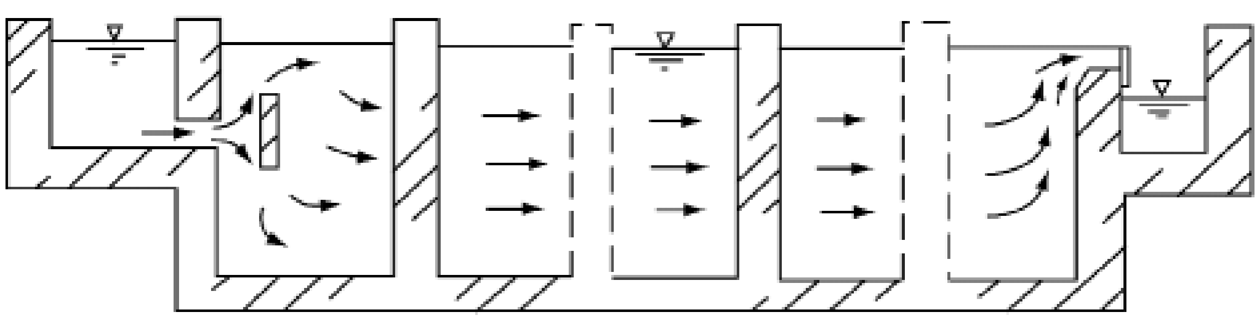

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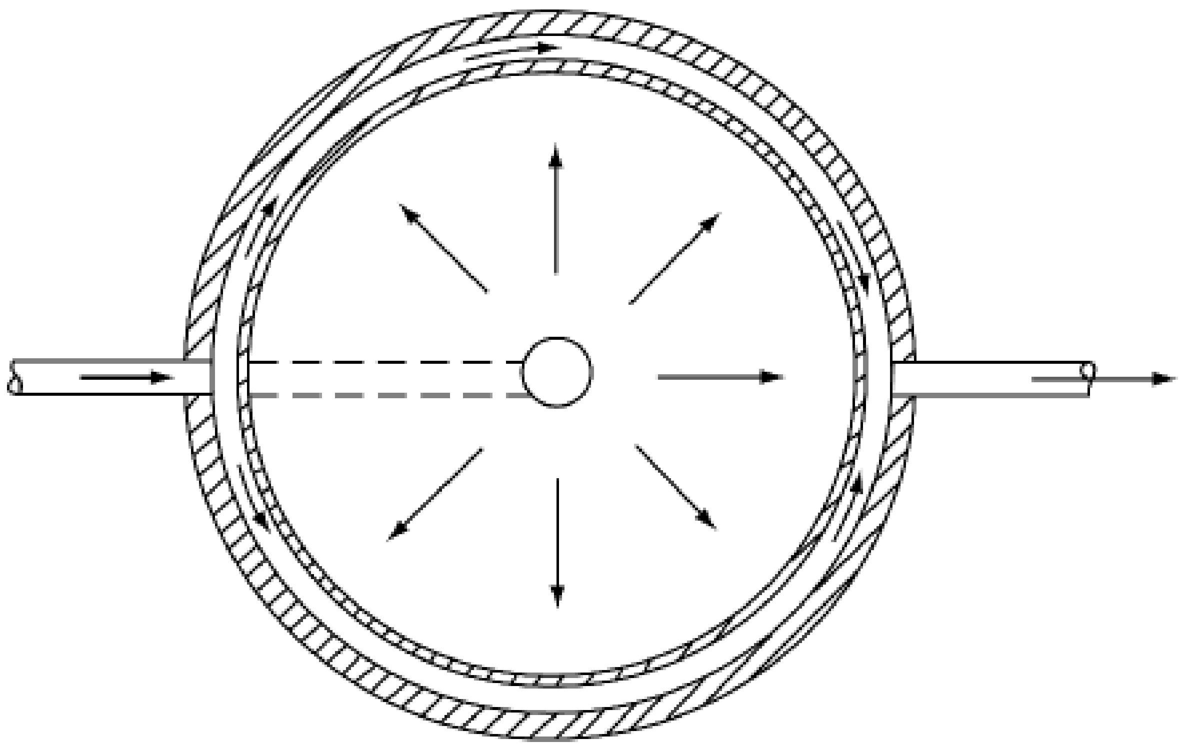
Poor Baffling Conditions - Circular Contact Basin



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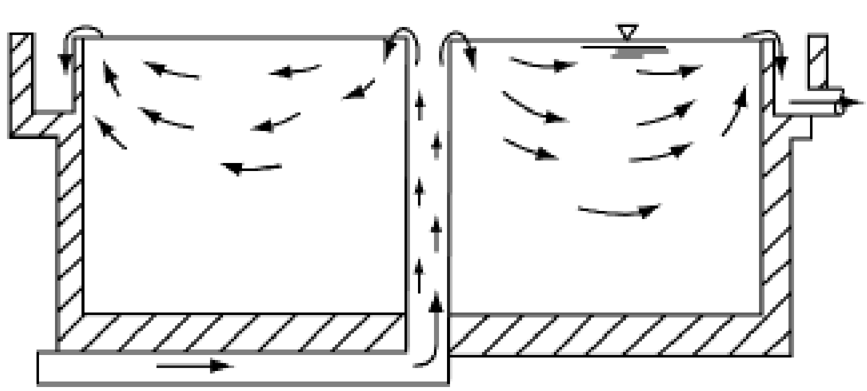

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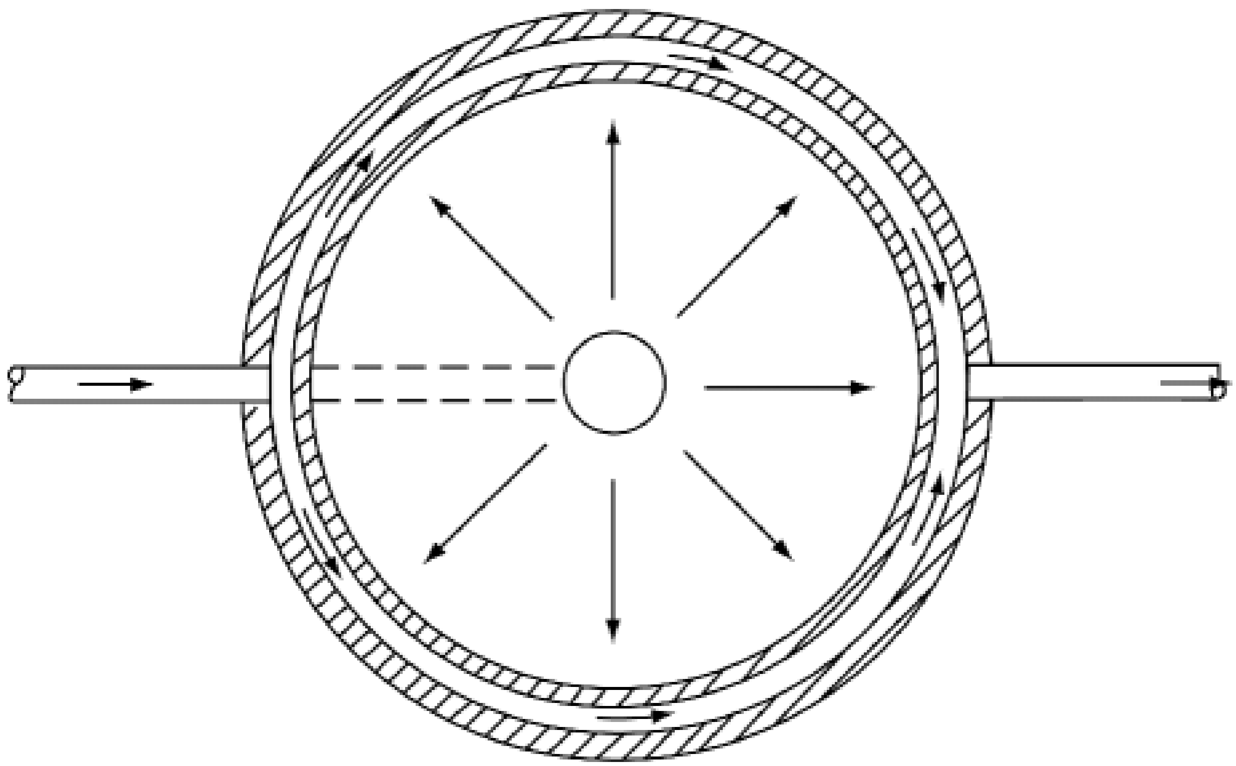
Average Baffling Conditions - Circular Contact Basin



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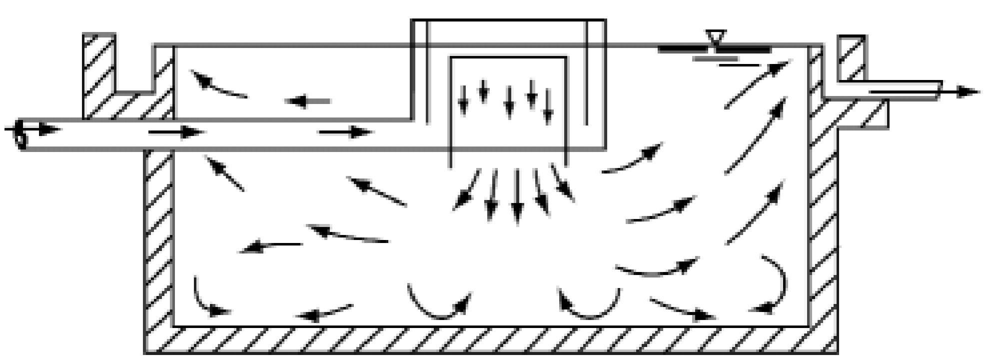
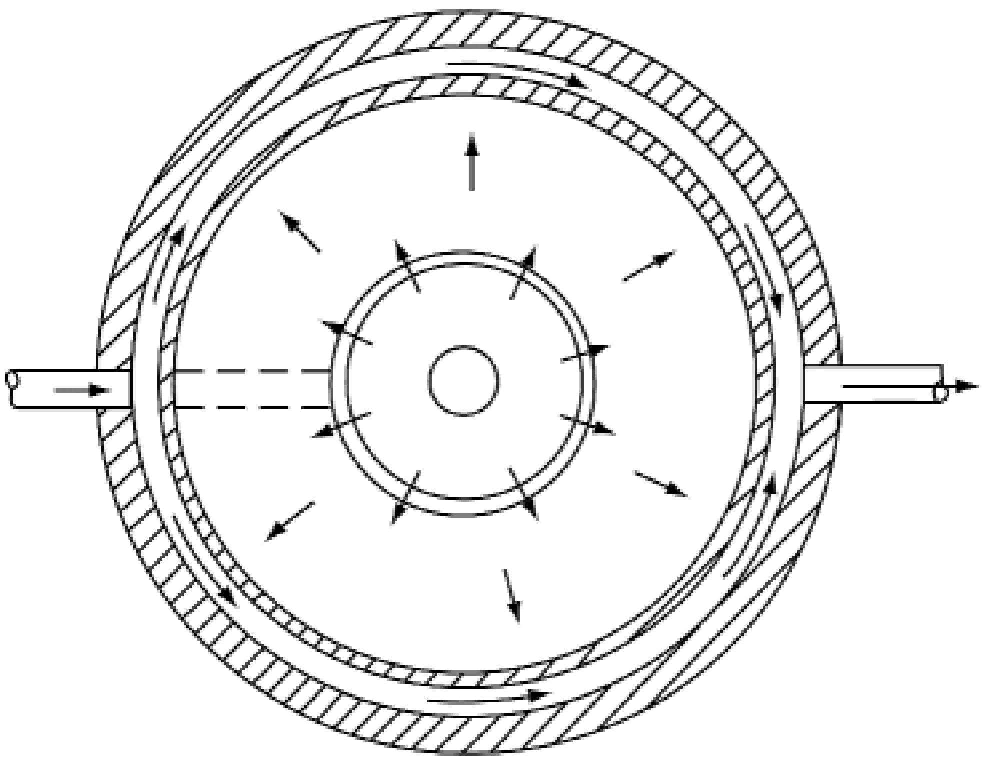


SECTION

Superior Baffling Conditions - Circular Contact Basin



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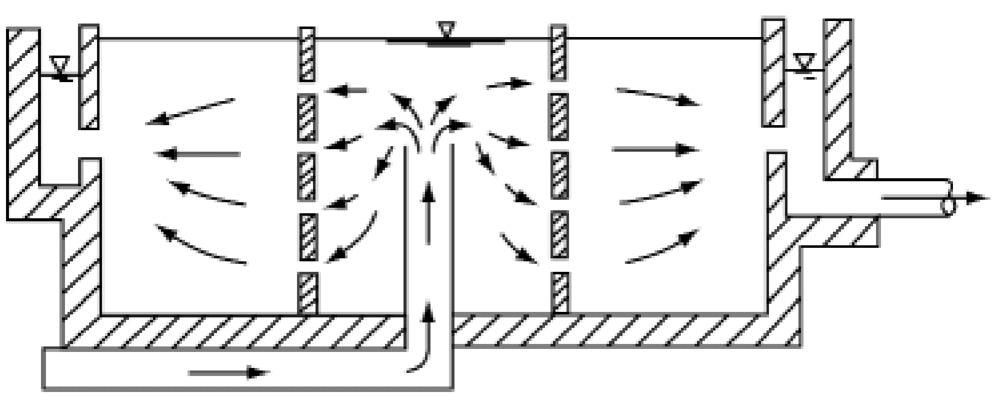

SECTION

APPENDIX D

BEST PROFESSIONAL JUDGEMENT (BPJ)

APPENDIX D

Best Professional Judgement (BPJ)

Section 2.2.1 makes reference to best professional judgement in determining the most suitable approach for achieving filtration credit. Examples of situations where BPJ could be applied are as follows:

1. The facility serves a small population;

   i. at an institution such as a school or community hall;
   ii. at an industry;
   iii. at a small development such as a mobile home park, work camp or provincial park; or
   iv. at a municpal development such as Village, Hamlet, bareland condominium or subdivision where the population is not over 250 persons.

2. The use of conventional treatment processes make the plant difficult to operate for the abilities of available operators and/or there is not sufficient technology support within a reasonable distance to effect repairs or replace parts.

3. The use of conventional treatment processes make the proposed plant too expensive to construct or operate.

APPENDIX E

TEST TIME CALCULATION FOR LOW-PRESSURE AIR TESTING OF INSTALLED PVC SEWER PIPE

(Reproduced from Uni-Bell PVC Pipe Association Standard UNI-B-6-90, Recommended Practice for Low-pressure Air Testing of Installed Sewer Pipe - May 1990)

1.1 TEST TIME CRITERIA

The Ramseier test time criteria requires that no test section shall be accepted if it loses more than Q cubic feet per minute per square foot of internal pipe surface area for any portion containing less than 625 square feet internal pipe surface area. The total leakage from any test section shall not exceed 625 Q cubic feet per minute.

1.2 ALLOWABLE AIR LOSS RATE

A Q value of 0.0015 cubic feet per minute per square foot shall be utilized to assure the Owner of quality pipe materials, good workmanship and tight joints.

1.3 TEST TIME CALCULATION

All test times shall be calculated using Ramseier's equation:

$$T = 0.085 \frac{DK}{Q}$$

Where:

Where:	T	=	Shortest time, in seconds, allowed for the air pressure to drop 1.0 psig,
	K	=	0.000419 DL, but not less than 1.0,
	Q	=	0.0015 cubic feet/minute/square feet of internal surface,
	D	=	Nominal pipe diameter in inches, and
	L	=	Length of pipe being tested in feet.

For more efficient testing of long test sections and/or sections of larger diameter pipes, a timed pressure drop of 0.5 psig may be used in lieu of the 1.0 psig timed pressure drop. If a 0.5 psig pressure drop is used, the appropriate required test times shall be exactly half as long as those obtained using Ramseier's equation for T cited above.

1.4 SPECIFIED TIME TABLES

To facilitate the proper use of this recommended practice for air testing, the following tables are provided. Table I contains the specified minimum times required for a 1.0 psig pressure drop from a starting pressure of at least 3.5 psig greater than the average back pressure of any groundwater above the pipe's invert. Table II contains specified minimum times required for a 0.5 psig pressure drop from a starting pressure of at least 3.5 psig greater than the average back pressure of any groundwater above the pipe's invert.

TABLE I

MINIMUM SPECIFIED TIME REQUIRED FOR A 1.0 PSIG PRESSURE DROP
FOR SIZE AND LENGTH OF PIPE INDICATED FOR Q = 0.0015

1 Pipe Diameter (in.)	2 Minimum Time (min: sec)	3 Length for Minimum Time (ft)	4 Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				100 ft	150 ft	200 ft	250 ft	300 ft	350 ft	400 ft	450 ft
4	3:46	597	.380 L	3:46	3:46	3:46	3:46	3:46	3:46	3:46	3:46
6	5:40	398	.854 L	5.40	5.40	5.40	5.40	5.40	5:40	5:42	6:24
8	7:34	298	1.520 L	7:34	7:34	7:34	7:34	7:36	8:52	10:08	11:24
10	9:26	239	2.374 L	9:26	9:26	9:26	9:53	11:52	13:51	15:49	17:48
12	11:20	199	3.418 L	11:20	11:20	11:24	14:15	17:05	19:56	22:47	25:38
15	14:10	159	5.342 L	14:10	14:10	17:48	22:15	26:42	31:09	35:36	40:04
18	17:00	133	7.692 L	17:00	19:13	25:38	32:03	38:27	44:52	51:16	57:41
21	19:50	114	10.470 L	19:50	26:10	34:54	43:37	52:21	61:00	69:48	78:31
24	22:40	99	13.674 L	22:47	34:11	45:34	56:58	68:22	79:46	91:10	102:33
27	25:30	88	17.306 L	28.51	43:16	57:41	72:07	86:32	100:57	115:22	129:48
30	28:20	80	21.366 L	35:37	53:25	71:13	89:02	106:50	124:38	142:26	160:15
33	31:10	72	25.852 L	43:05	64:38	86:10	107:43	129:16	150:43	172:21	193:53
36	34:00	66	30.768 L	51:17	76:55	102:34	128:12	153:50	179:29	205:07	230:46

TABLE II

MINIMUM SPECIFIED TIME REQUIRED FOR A 0.5 PSIG PRESSURE DROP
FOR SIZE AND LENGTH OF PIPE INDICATED FOR Q = 0.0015

1 Pipe Diameter (in.)	2 Minimum Time (min: sec)	3 Length for Minimum Time (ft)	4 Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				100 ft	150 ft	200 ft	250 ft	300 ft	350 ft	400 ft	450 ft
4	1:53	597	.190 L	1:53	1:53	1:53	1:53	1:53	1:53	1:53	1:53
6	2:50	398	.427 L	2:50	2:50	2:50	2:50	2:50	2:50	2:51	3:12
8	3:47	298	.760 L	3:47	3:47	3:47	3:47	3:48	4:26	5:04	5:42
10	4:43	239	1.187 L	4:43	4:43	4:43	4:57	5:56	6:55	7:54	8:54
12	5:40	199	1.709 L	5:40	5:40	5:42	7:08	8:33	9:58	11:24	12:50
15	7:05	159	2.671 L	7:05	7:05	8:54	11:08	13:21	15:35	17:48	20:02
18	8:30	133	3.846 L	8:30	9:37	12:49	16:01	19:14	22:26	25:38	28:51
21	9:55	114	5.235 L	9:55	13:05	17:27	21:49	26:11	30:32	34:54	39:16
24	11:20	99	6.837 L	11:24	17:57	22:48	28:30	34:11	39:53	45:35	51:17
27	12:45	88	8.653 L	14:25	21:38	28:51	36:04	43:16	50:30	57:42	46:54
30	14:10	80	10.683 L	17:48	26:43	35:37	44:31	53:25	62:19	71:13	80:07
33	15:35	72	12.926 L	21:33	32:19	43:56	53:52	64:38	75:24	86:10	96:57
36	17:00	66	15.384 L	25:39	38:28	51:17	64:06	76:55	89:44	102:34	115:23