

The Alberta Gazette

Part I

Vol. 109

Edmonton, Friday, March 15, 2013

No. 05

GOVERNMENT NOTICES

Agriculture and Rural Development

Form 15

(Irrigation Districts Act)
(Section 88)

Notice to Irrigation Secretariat:
Change of Area of an Irrigation District

On behalf of the **Bow River Irrigation District**, I hereby request that the Irrigation Secretariat forward a certified copy of this notice to the Registrar for Land Titles for the purposes of registration under section 22 of the *Land Titles Act* and arrange for notice to be published in the Alberta Gazette.

The following parcels of land should be **added** to the irrigation district and the notation added to the certificate of title:

LINC Number	Short Legal Description as shown on title	Title Number
0022 326 318	S.E. 3-17-18-W4M	111 138 282 +1
0022 317 861	N.W. 3-17-18-W4M	101 124 590 +3
0022 317 721	S.W. 16-17-18-W4M	101 124 590 +2
0026 394 809	N.W. 9-15-18-W4M	951 106 092
0035 420 041	N.E. 11-19-22-W4M	121 293 875
0022 259 279	N.E. 34-13-19-W4M	051 164 880
0022 259 303	N.W. 35-13-19-W4M	051 164 880 +1
0032 987 852	S.E. 4-15-19-W4M	131 036 835

I certify the procedures required under part 4 of the *Irrigation Districts Act* have been completed and the area of the **Bow River Irrigation District** should be changed according to the above list.

Rebecca Fast, *Office Administrator,*
Irrigation Secretariat.

Culture

Hosting Expenses Exceeding \$600.00 *For the Period July 1 to September 30, 2012*

Function: Edmonton Southeast Business Leaders Luncheon

Date: January 26, 2012

Amount: \$1,776.50 An additional \$1,782 paid and reported by Education.

Purpose: Meeting with the business community to discuss opportunities for involvement and leadership in Education and Culture initiatives.

Location: The Palace Banquet Facility, Edmonton, AB

BU #: 022

Function: Ministerial Conference on the Canadian Francophonie – reception and banquet

Date: June 26, 2012

Amount: \$9,376.00

Purpose: To hold a reception and banquet with the Alberta francophone community.

Location: La Cite Francophone, Edmonton, AB

BU #: 022

Function: Ministerial Conference on the Canadian Francophonie – breakfast, lunch and coffee

Date: June 27, 2012

Amount: \$13,997.25

Purpose: To provide breakfast, lunch and coffee breaks to the attendees of the Conference.

Location: Westin Hotel, Edmonton, AB

BU #: 022

Function: Head-Smashed-In Buffalo Jump World Heritage Site 25th Anniversary Celebrations

Date: July 25, 2012

Amount: \$2,260.00

Purpose: To provide celebration refreshments for dignitaries, guests, visitors, staff and contractors involved in the site's development and operations.

Location: Head-Smashed-In Buffalo Jump World Heritage Site, Fort Macleod, AB

BU #: 024

Function: 2012 Harvest Festival

Date: September 1 and 2, 2012

Amount: \$1,440.48

Purpose: To provide lunch, snacks and refreshments for 100 volunteers assisting with the two-day 2012 Harvest Festival event.

Location: Reynolds-Alberta Museum, Wetaskiwin, AB

BU #: 024

Function: Alberta Foundation for the Arts (AFA) - Arts Community Reception

Date: September 6, 2012

Amount: \$1,200.00

Purpose: To provide an opportunity for AFA board members to network with stakeholders in the region.

Location: Elevation Gallery, Canmore, AB

BU #: 027

Function: Toronto International Film Festival - Networking Reception

Date: September 11, 2012

Amount: \$3,179.22

Purpose: Networking Reception for industry stakeholders and key contacts during the Toronto International Film Festival. This reception was co-hosted by the Nova Scotia Film Commission.

Location: The Drake Hotel, Toronto, ON

BU #: 022

Function: Alberta Culture Days 2012

Date: September 28, 29 and 30, 2012

Amount: \$3,796.13

Purpose: To promote and raise public awareness and increase attendance at Alberta Culture Days events.

Location: Various locations throughout Alberta

BU #: 022

Function: Head-Smashed-In Buffalo Jump 25th Anniversary Staff Homecoming Event and BBQ

Date: September 29, 2012

Amount: \$3,500.00

Purpose: To host a homecoming celebration for those who have worked at the site over the past 25 years.

Location: Head-Smashed-In Buffalo Jump World Heritage Site, Fort Macleod, AB

BU #: 024

Notice of Receipt of a Repatriation Application

(Blackfoot First Nations Sacred Ceremonial Objects Repatriation Regulation)

Pursuant to the *First Nations Sacred Ceremonial Objects Repatriation Act (FNSCORA)* and the *Blackfoot First Nations Sacred Ceremonial Objects Repatriation Regulation* made under that Act, the Province of Alberta as represented by the Minister of Culture hereby gives notice of receipt of a repatriation application from a member of the Blood Tribe for an object in the Government of Alberta's collections at the Royal Alberta Museum.

The application is for the repatriation of a Pipe (H66.107.2 A-B) originating with the Blood Tribe. The Pipe originated with the Brave Dog Leader's Bundle and was purchased by the Royal Alberta Museum in 1966.

Any person with an interest in these objects, who either wishes to make written representations to the Minister relating to this application or to submit an application of their own, must do so within 30 days after publication of this notice in The Alberta Gazette. For further information, an interested person should contact the Aboriginal Liaison Officer at the Royal Alberta Museum, 12845-102 Avenue, Edmonton, Alberta T5N 0M6. The Aboriginal Liaison Officer can also be reached by telephone at (780) 453-9155 or by fax at (780) 454-6629 or email- judy.half@gov.ab.ca. Repatriation of the above objects to the Mookaakin Heritage Foundation, as the designated representative of the Blood Tribe, may proceed after the 30 day period if the object listed in the application is determined to meet the criteria of a sacred ceremonial object as defined in FNSCORA, no additional applications or contrary written representations have been made, and if the Minister considers repatriation to be appropriate.

Energy

Declaration of Withdrawal from Unit Agreement

(Petroleum and Natural Gas Tenure Regulations)

The Minister of Energy on behalf of the Crown in Right of Alberta hereby declares and states that the Crown in right of Alberta has withdrawn as a party to the agreement entitled "Scandia Gas Unit No. 1" effective February 28, 2013.

Claudia Cooper, *for Minister of Energy*.

Production Allocation Unit Agreement

(Mines and Minerals Act)

Notice is hereby given, pursuant to section 102 of the Mines and Minerals Act, that the Minister of Energy on behalf of the Crown has executed counterparts of the agreement entitled "Production Allocation Unit Agreement – Lloydminster Agreement No. 7" and that the Unit became effective on November 1, 2012.

EXHIBIT "A" - PART I
ATTACHED TO AND MADE PART OF AN AGREEMENT ENTITLED
"Lloydminster Agreement No. 7"

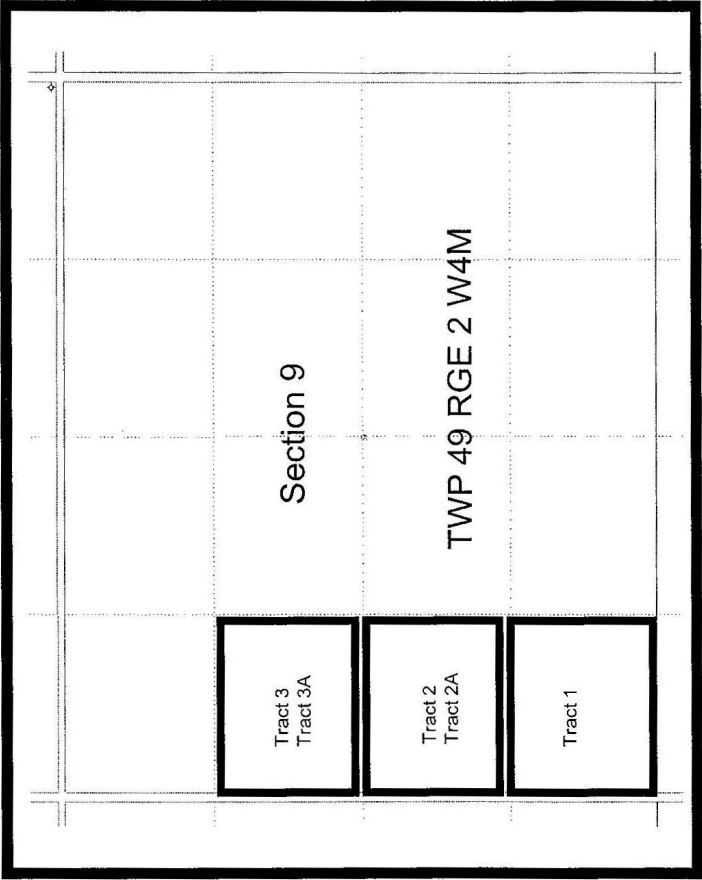
Tract No.	Land Description (W4M)	Lease Number	Royalty Owner	Tract Participation (%)	Working Interest Owner	Share of Working Interest (%)	Share of Tract Participation (%)
1	LSD 4 of Section 9-49-2W4M	PanCanadian C-1246	Husky Oil Operations Limited	33.333	Husky Oil Operations Limited	100	33.333
2	LSD 5 of Section 9-49-2W4M	0479110201	Crown	10.159	Husky Oil Operations Limited	100	10.159
2A	LSD 5 of Section 9-49-2W4M	PanCanadian C-1246	Husky Oil Operations Limited	23.175	Husky Oil Operations Limited	100	23.175
3	LSD 12 of Section 9-49-2W4M	0479110201	Crown	20.156	Husky Oil Operations Limited	100	20.156
3A	LSD 12 of Section 9-49-2W4M	PanCanadian C-1246	Husky Oil Operations Limited	13.177	Husky Oil Operations Limited	100	13.177
Total			Crown Husky	30.315 69.685			
				100.000			

Working Interest Owners:

HUSKY OIL OPERATIONS LIMITED 100%

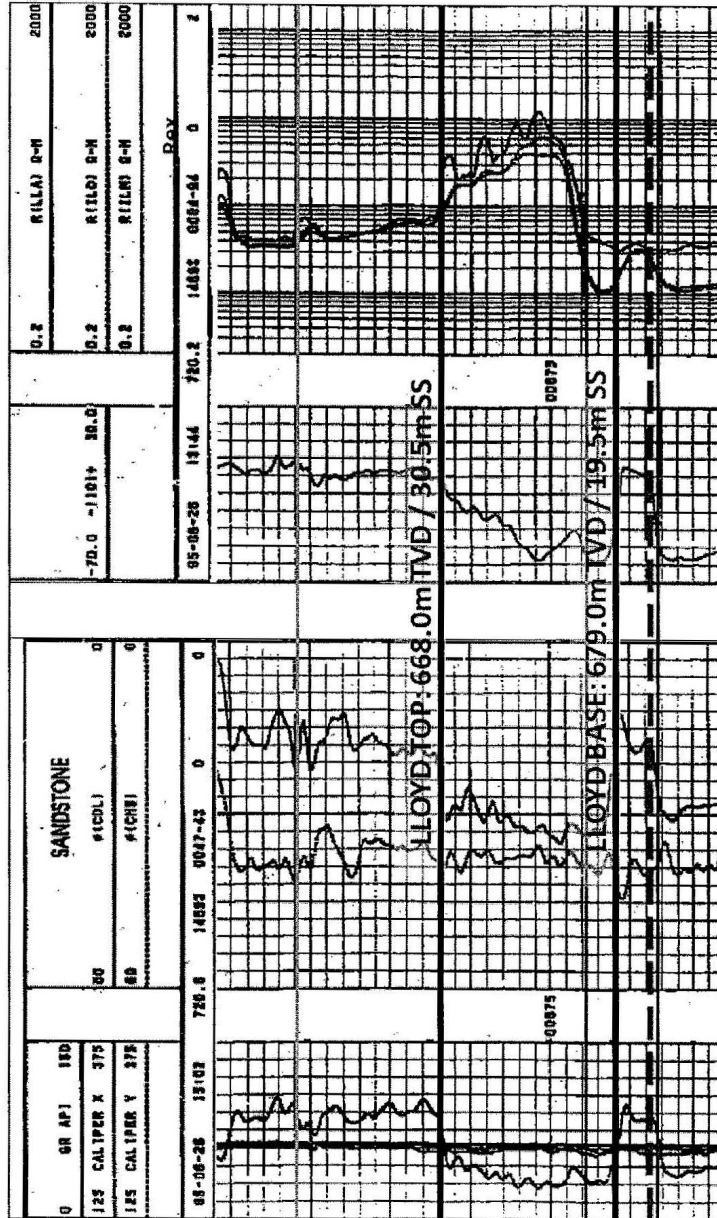
Effective as of the Effective Date

EXHIBIT "B"
ATTACHED TO AND MADE PART OF AN AGREEMENT ENTITLED LLOYDMINSTER AGREEMENT NO. 7



Effective as of the Effective Date

EXHIBIT "C"
 ATTACHED TO AND MADE PART OF AN AGREEMENT ENTITLED
 Lloydminster Agreement No. 7
 00/12-09-049-02W4/00



Production Allocation Unit Agreement

(Mines and Minerals Act)

Notice is hereby given, pursuant to section 102 of the Mines and Minerals Act, that the Minister of Energy on behalf of the Crown has executed counterparts of the agreement entitled "Production Allocation Unit Agreement – Swimming McLaren Agreement Unit No. 2" and that the Unit became effective on December 1, 2012.

EXHIBIT "A" - PART I
ATTACHED TO AND MADE PART OF AN AGREEMENT ENTITLED
"Swimming McLaren Agreement No. 2"

Tract No.	Land Description (W4M)	Lease Number	Royalty Owner	Tract Participation (%)	Working Interest Owner	Share of Working Interest (%)	Share of Tract Participation (%)
1	TWP 052 RGE 07 W4M PTN. SW ¼ Sec 15 (Not under waters Lake No. 2)	P&NG Lease dated Nov 7, 2006	FH Big Bend Alberta Ltd.	5.03	Husky Oil Operations Limited	100	5.03
1a	TWP 052 RGE 07 W4M PTN. SW ¼ Sec 15 (Under Waters Lake No. 2)	04010708009	Crown	2.58	Husky Oil Operations Limited	100	2.58
2	TWP 052 RGE 07 W4M NW ¼ Sec 10	26864	Crown	43.60	Husky Oil Operations Limited	100	43.60
3	TWP 052 RGE 07 W4M NE ¼ Sec 10	26864	Crown	48.79	Husky Oil Operations Limited	100	48.79

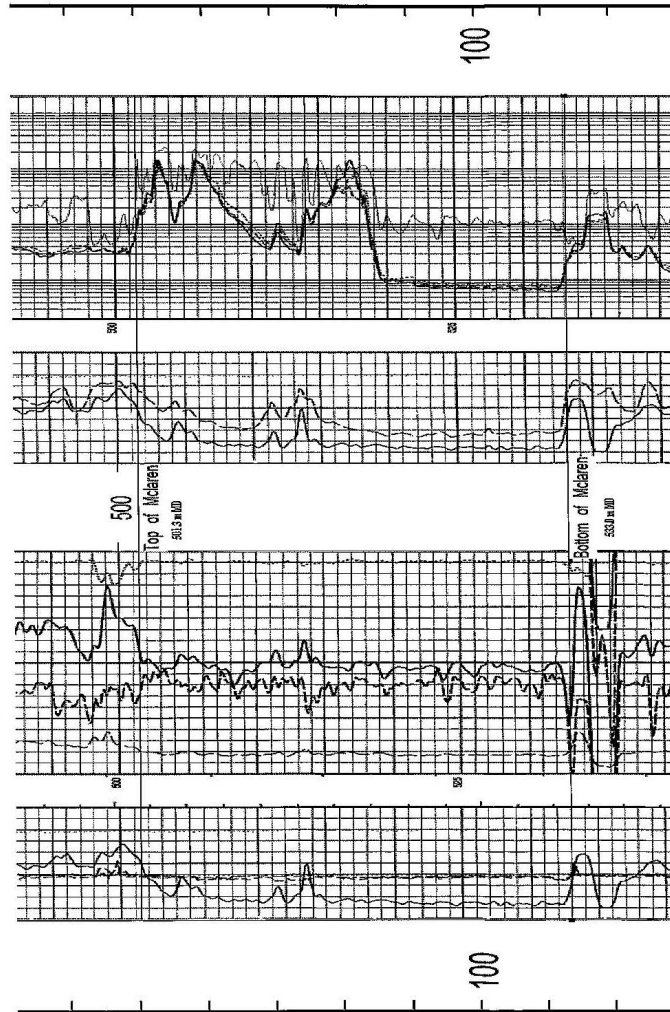
Working Interest Owners:

Husky Oil Operations Limited

100%

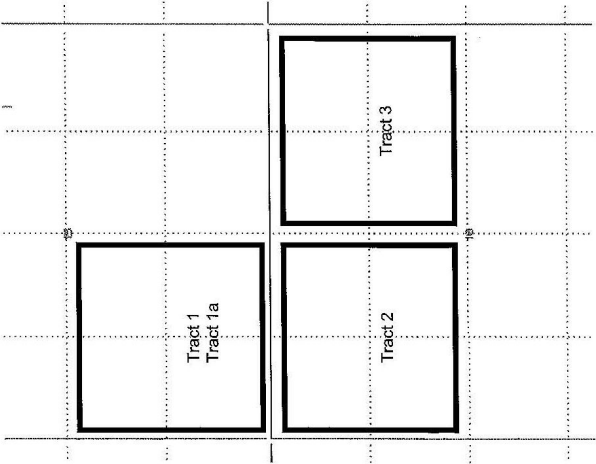
Effective as of the Effective Date

EXHIBIT "C"
ATTACHED TO AND FORMING PART OF AN AGREEMENT ENTITLED
SWIMMING MCLAREN AGREEMENT NO. 2



09/04-15-052-07W4/0

Exhibit "B"
ATTACHED TO AND MADE PART OF AN AGREEMENT ENTITLED SWIMMING MCLAREN AGREEMENT NO.2



Effective as of the Effective Date

Environment and Sustainable Resource Development

Notice of Variation Order 27-2012

Commercial Fishing Seasons

The close times and quotas set out in Schedule 8 to the Alberta Fishery Regulations, 1998 in respect of the waters listed in the Schedule to this Notice have been varied by Variation Order 27-2012 by the Director of Fisheries Management in accordance with section 3 of the Alberta Fishery Regulations, 1998.

Where fishing with gill nets is permitted during an open season established by the Order, the gill net mesh size has been specified in the Order.

Pursuant to Variation Order 27-2012 commercial fishing is permitted in accordance with the following schedule.

SCHEDULE PART 1

Item - 3

Column 1 Waters – In respect of: (55.1) Lesser Slave Lake (74-11-W5) – that portion east of the eastern boundary of Range 10, west of the 5th Meridian

Column 2 Gear - Gill net not less than 127 mm mesh

Column 3 Open Time - A. In respect of Lesser Slave Lake excluding the following portions: a) that portion west of a line drawn from the point of land in the SW8-74-8-W5 (11 U E 614260, N 6139740) locally known as the west side of Willow Point at the mouth of the Assinneau River, to the point of land in the SW26-75-8-W5 (11 U E 617677, N 6154610) locally known as Big Stony Point; b) that portion that is greater than 30 feet (9.15 m) in depth south and east of a line drawn from the point of land in SW 26-73-7-W5M (UTMs: E 628702 / N 6135803) locally known as Nine Mile Point, to the mouth of Lily Creek located in NW 14-74-6-W5M (UTMs: E 638347 / N 6143144); and c) that portion that is greater than 50 feet (15.24 m) in depth that is north and west of the line described in point (b) and east of the line described in point (a): 08:00 hours January 25, 2013 to 16:00 hours **March 31**, 2013. B. In respect of all other waters: Closed.

Column 4 Species and Quota - 1) Lake whitefish: 30,622 kg; 2) Walleye: 3,461 kg; 3) Yellow perch: 100 kg; 4) Northern pike: 2,417 kg; 5) Tullibee: 5,000 kg; 6) Lake trout: 1 kg.

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

Part 3 Wastewater Systems Standards for Performance and Design
of a Total of 5 Parts

March 2013

Part 3 WASTEWATER SYSTEM STANDARDS FOR PERFORMANCE AND DESIGN
March 2013

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Additional Parts published separately are:

Part 1 Standards for Municipal Waterworks

Part 2 Guidelines for Municipal Waterworks

Part 4 Wastewater Systems Guidelines for Design, Operating and Monitoring Requirements

Part 5 Stormwater Management Guidelines

FOREWORD TO PART 3 WASTEWATER SYSTEMS STANDARDS
FOR PERFORMANCE AND DESIGN (2013)

Alberta Environment and Sustainable Resource Development (AESRD) has the regulatory mandate, in accordance with the Environmental Protection and Enhancement Act and Regulations, for the Drinking Water and Wastewater Programs serving large public systems in Alberta. AESRD 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. AESRD's objective is to develop comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable.

Since publication of the last revision of the Standards and Guidelines, Alberta Environment and Sustainable Resource Development has embarked on a process of "decoupling" the various components of the January 2006 document into functionally-associated sections to aid those using the document. This process started with the publication of the January 2006 version of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in the Alberta Gazette. A program of separating the component parts of this document is under way and new parts will eventually replace the corresponding sections in the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems. Until the process of "decoupling" is completed with new "Parts" the existing sections of the 2006 Standards and Guidelines document will remain in operation. This part (Part 3 – Wastewater Systems Standards for Performance and Design) has all activities and / or system components that relate to protection of the environmental and human health. System components that are guidance to best practices are included under "Part 4 – Wastewater Systems Guidelines for Design, Operating and Monitoring Requirements".

The system owners / utilities are responsible for meeting AESRD's regulatory requirements and for the collection, treatment and disposal or use of treated wastewater. They are also responsible for maintaining the wastewater collection system up to the service connection. Engineering consultants and / or the system owners / utilities are responsible for the detailed project design and satisfactory construction and operation of the waterworks and wastewater systems.

In accordance with the Wastewater and Storm Drainage Regulation (119/1993) a wastewater system and storm drainage system will be designed so that it meets, as a minimum, the performance standards and design requirements set out in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AESRD, as amended or replaced from time to time, or, any other standards and design requirements specified by the Regional Director.

AESRD last revised its Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in January 2006. The changes in this version from the January 2006 Standards include correction of some clerical error, updating Tables 3.6 and 3.7 to be consistent with Subdivision and Development Regulation, and the renumbering of Sections 3 Wastewater Systems Performance Standards and Section 4 Wastewater Systems Design Standards into a single document. The original 2006 Appendix F appears in this document as Appendix 3-A.

This part details all the critical elements of Wastewater Systems Performance and Design. These are mandatory requirements with which owners are required to comply. If at any time,

the performance standards are compromised, the approval / registration holder shall immediately report to the Regional Director, either:

1. by telephone at 1-800-222-6514; or
2. by a method:
 - a. in compliance with the release reporting provisions in the Act and the regulations, or
 - b. as authorized in writing by the Regional Director.

DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AESRD	-	Alberta Environment and Sustainable Resource Development
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 °C
CFID	-	Continuous feed and intermittent discharge
DAF	-	Dissolved Air Flotation
DBP	-	Disinfection By-product
DCS	-	Distributed Control System
DO	-	Dissolved Oxygen
DWSP	-	Drinking Water Safety Plan
DOC	-	Dissolved Organic Carbon
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWUDI	-	Groundwater under the direct influence of surface water
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
IFID	-	Intermittent feed and intermittent discharge
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 °C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
USEPA	-	United States Environmental Protection Agency
UV	-	Ultraviolet
WHO	-	World Health Organization

Average daily design flow - The product of the following:

- design population of the facility, and
- 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.

Maximum daily design flow (water) - Maximum three 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.

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

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

Potable water - As defined in the EPEA. Other domestic purposes in the EPEA definition include water used for personal hygiene, e.g. bathing, showering, washing, etc.

Septage - The liquid, solid or semisolid material removed from septic tanks, portable toilets, and holding tanks that receive sewage from domestic sources. This does not include wastes from grease traps, industrial or commercial processes.

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.

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

Note: All concentrations expressed in milliequivalents per litre

Surface water - Water in a watercourse.

Watercourse - As defined in the EPEA.

3.0 Wastewater Systems - Performance and Design Standards

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 AESRD standards (Section 3.4.1). 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 AESRD.

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	25 mg/L	Composite	Monthly average of daily samples
	TSS	25 mg/L	Composite	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 3.5 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:

- 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.
- Sampling frequencies are based on continuous discharge of effluent to a body of water.
- Facilities producing effluent with nitrogenous oxygen demand may be required to monitor for TBOD. The need for TBOD monitoring, and subsequent limit on NH₃-N will be assessed on a site-specific basis.
- 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.
- 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.
- 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.
- See Table 3.5 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 months of storage; or
2. Secondary treatment with or without storage.

3.2.1.2 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.

3.3 Wastewater Collection

3.3.1 Sanitary Sewers

3.3.1.1 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.

3.3.1.2 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 3.4.

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 AESRD. (See Sections 3.3.1.3 and 3.3.1.4).

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.

3.3.1.3 Minimum Flow Depths

As per Section 3.3.1.2, 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 AESRD that any additional sewer maintenance required as a result of reduced slopes will be provided.

3.3.1.4 Solids Deposition

The pipe diameter and slope shall be selected to obtain practical velocities to minimize settling problems. As per Section 3.3.1.2, 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 AESRD.

3.3.1.5 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.

3.3.1.6 Frost Protection

Frost protection criteria for sewers is the same as for water mains (see Section 1.9.2.1 for details).

3.3.1.7 Cross Connections

Cross connection prevention is the same as for water supply (see Section 1.9.3 for details).

TABLE 3.4
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

3.3.2 Wastewater Pump Stations

3.3.2.1 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.

3.3.2.2 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.

Safety Ventilation

1. General

Adequate ventilation shall be provided for all pump stations. Ventilation systems, including fresh air intake louvers / 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 4.3.3.

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.

3.3.2.3 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.

3.3.2.4 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.

3.3.2.5 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 3.4.3.4 (3). In-line backflow preventers are not acceptable.

3.3.3 Screens and Grit Removal Facilities

3.3.3.1 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.

3.3.4 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, Regulation and Code*.

3.4 Wastewater Treatment and Disposal

3.4.1 Wastewater Lagoons

3.4.1.1 General Requirements

The minimum design standards for wastewater lagoons are a function of the average daily design flow. AESRD 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. Addition of a mechanical aeration system or mixing units to a wastewater lagoon designed to meet the minimum standards for wastewater lagoons will not change the system to an aerated lagoon or mechanical system. For aerated lagoon systems see Section 3.4.4.

3.4.1.2 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 3.0 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 m³. Therefore, the minimum average daily design flow at which an anaerobic cell is practical is approximately 250 m³/d.

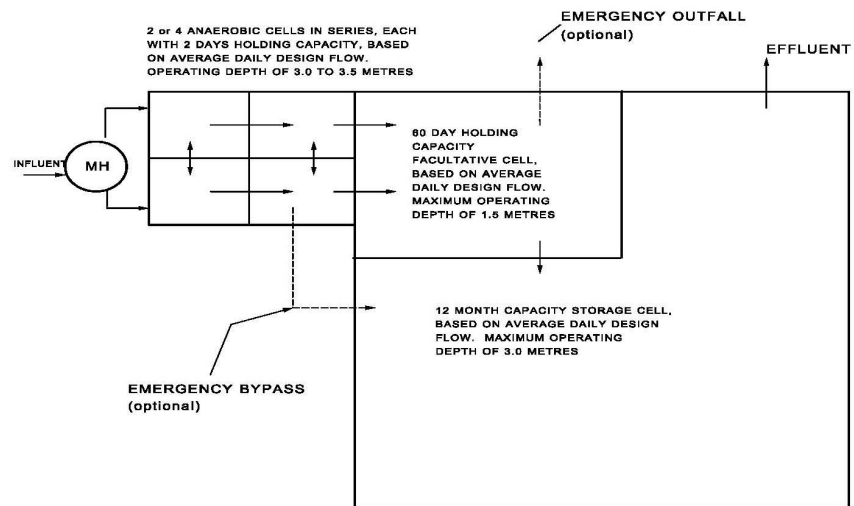
When design flows warrant the provision of anaerobic cells, there shall be a minimum of two and a maximum of four cells operated in series. The number of anaerobic cells is a function of design daily flows as illustrated in Table 3.5 and Figure 3.1.

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 3.1.

FIGURE 3.1
Typical Wastewater Lagoons



**TABLE 3.5
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.

3.4.1.3 Design Considerations

There are a number of siting and design features which must be considered in order to protect public 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 3.6 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 3.6
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
A "building site"*** for school, hospital, food establishment or residential use	300

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

** "Building site" means a portion of the land on which a building exists, or can or may be constructed

Note:

- Minimum setback distance from the "working area" of the wastewater lagoon to a "building site" on the property of a "privately owned development" which the lagoon serves may be reduced to 100 m.
- Setback distances may be varied with the written consent of the "Director".

3.4.1.4 Seepage Control

The following sections are a summary of the Alberta Environment and Sustainable Resource Development 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:

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

governed by Darcy's Law, shall be calculated using the following equation:

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 five (5) boreholes or one (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:

- a. 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 side slope 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;
- b. Compacted Clay Liners shall have a minimum thickness of 0.6 m on the bottom and 1.2 m on the side slope (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;
- c. 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;
- d. 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 sub grades. The spray-on bitumen covering the hydraulic asphalt concrete or soil asphalt surface should provide a uniform 2 mm thick membrane; and
- e. 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 side slopes 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:

- a. 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.
- b. 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.
- c. Water samples shall be analyzed for routine water chemistry, total Kjeldahl nitrogen and chemical oxygen demand.
- d. 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.

3.4.1.5 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

- a. Embankment tops shall have a minimum width of 3 m to provide a driving surface for maintenance vehicles;
- b. 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;
- c. 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;
- d. 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
- e. 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 4.2.4.2 (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 is required to transport flows to the wastewater lagoons, and the system has a capacity greater than 500 m³/d, a portable or permanent flow measuring device shall be provided at the inlet of the wastewater lagoons.

3.4.2 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.

3.4.2.1 System Components and Configuration

For systems with average daily design flows of less than 250 m³, 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 m³/d, the evaporation cell shall be preceded by two anaerobic or 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 three years of storage capacity based on average daily design flows. In practice, more than three 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.

3.4.2.2 Design Considerations

Design requirements outlined for wastewater lagoons in Sections 3.4.1.3, 3.4.1.4 and 3.4.1.5 also apply to evaporation lagoons.

3.4.3 Mechanical Wastewater Treatment

3.4.3.1 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.

3.4.3.2 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 3.7 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 3.4.1.3 (3), (4), (5) and (6) respectively.

**TABLE 3.7
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 lagoon to:	
The property line of the land where the operating mechanical treatment plant or aerated lagoons 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
A "building site"*** for school, hospital, food establishment or residential use	300

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

** "Building site" means a portion of the land on which a building exists, or can or may be constructed

Note:

- Minimum setback distance from the "working area" of the operating mechanical treatment plant or aerated lagoons to a "building site" on the property of a "privately owned development" which the lagoon serves may be reduced to 100 m.
- Setback distances may be varied with the written consent of the "Director".

3.4.3.3 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 handrails 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; and
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.

3.4.3.4 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:

- a. lavatory;
- b. water closet;
- c. laboratory sink (with vacuum breaker);
- d. shower;
- e. drinking fountain;
- f. eye wash fountain; and
- g. 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.

3.4.4 Aerated Lagoons

3.4.4.1 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.

3.4.4.2 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.

3.4.4.3 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 | - | 3.4.1.3 (3) |
| 2. | Fencing | - | 3.4.1.3 (4) |
| 3. | Signs | - | 3.4.1.3 (5) |
| 4. | Access | - | 3.4.1.3 (6) |
| 5. | Surface Runoff | - | 3.4.1.3 (7) |
| 6. | Seepage Control | - | 3.4.1.4 (1-4) |
| 7. | Construction Features- | | 3.4.1.5 (1-4) |
| 8. | Setback Distances | - | 3.4.3.2 (1) |

3.4.5 Disinfection

3.4.5.1 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. When chlorine or one of its compounds is used in the treatment of wastewater, the average concentration of total residual chlorine in the effluent discharged to the environment shall not exceed 0.02 mg/L. The total residual chlorine sampling frequency and averaging period shall be consistent with the CBOD requirements.

3.4.6 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 2.3.9, "Water Treatment Chemicals" for information on Labels and Material Safety Data Sheets and Storage Handling.

3.4.7 Effluent Disposal

3.4.7.1 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. Prior to each discharge, the discharge course and receiving environment should be inspected to identify and address constraints. 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 the 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 manmade or intermittent natural drainage course to convey the continuous discharge for a short distance to the receiving watercourse.

3.4.7.2 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 AESRD.

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 and Sustainable Resource Development - 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; and
- 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.

3.4.8 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 4.4).

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. Sanitary landfilling;
2. Incineration;
3. Permanent lagoon storage; and
4. Land application.

Of these disposal methods, the only ways of re-utilizing and conserving the useful constituents in biosolids are through application to land or generation of energy by incineration (provided a positive energy balance is achieved). With proper management, these methods are 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 AESRD. 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.

Although a potential soil conditioner and fertilizer, biosolids may also contain pathogenic organisms and quantities of elements and chemicals that can adversely affect crop quality and yield and result in pollution of surface and groundwaters, if improperly managed. These guidelines provide management requirements that will maximize the benefits associated with land application of biosolids while minimizing the potential risks.

3.4.9 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, Regulation and Code*.

APPENDIX 3-A

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: 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

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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	64:54
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

3-A-3

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

Part 4 Wastewater Systems Guidelines for Design, Operating and Monitoring
of a Total of 5 Parts

March 2013

**Part 4 WASTEWATER SYSTEMS GUIDELINES FOR DESIGN, OPERATING,
AND MONITORING
March 2013**

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Additional Parts published separately are:

Part 1 Standards for Municipal Waterworks
 Part 2 Guidelines for Municipal Waterworks
 Part 3 Wastewater Systems Standards for Performance and Design
 Part 5 Stormwater Management Guidelines

**FOREWORD TO PART 4 WASTEWATER SYSTEMS GUIDELINES FOR DESIGN,
OPERATING AND MONITORING (2013)**

Alberta Environment and Sustainable Resource Development (AESRD) has the regulatory mandate, in accordance with the Environmental Protection and Enhancement Act and Regulations, for the Drinking Water, Wastewater and Storm Drainage serving large public systems in Alberta. AESRD 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. AESRD's objective is to develop comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable.

Since publication of the last revision of the Standards and Guidelines, Alberta Environment and Sustainable Resource Development has embarked on a process of "decoupling" the various components of the January 2006 document into functionally-associated sections to aid those using the document. This process started with the publication of the January 2006 version of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in the Alberta Gazette. A program of separating the component parts of this document is under way and new parts will eventually replace the corresponding sections in the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems. Until the process of "decoupling" is completed with new "Parts" the existing sections of the 2006 Standards and Guidelines document will remain in operation. This Part (Part 4) details system components that are guidance to best practices in providing well managed wastewater systems and should be read in conjunction with Part 3 – Wastewater Systems Standards for Performance and Design (2013).

The system owners / utilities are responsible for meeting AESRD's regulatory requirements and for the collection, treatment and disposal or use of wastewater. They are also responsible for maintaining the wastewater collection system up to the service connection. Engineering consultants and / or the system owners / utilities are responsible for the detailed project design and satisfactory construction and operation of the waterworks and wastewater systems.

In accordance with the Wastewater and Storm Drainage Regulation (119/1993) a wastewater system and storm drainage will be designed so that it meets, as a minimum, the performance standards and design requirements set out in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AESRD, as amended or replaced from time to time, or, any other standards and design requirements specified by the Regional Director. AESRD last revised its Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in January 2006.

This document entitled Part 4, is intended to provide general guidance on how to achieve a certain level of wastewater system performance or reliability. Good engineering and best management practices are included in this Part. These are not mandatory requirements but they establish what is expected when the system owner / utility owners applies for an approval.

This version of Part 4 – Wastewater System Guidelines for Design, Operation and Monitoring includes Sections 5 and 7 of the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems with some corrections and the addition of Section 4.5 outlining guidelines for septage management systems.

DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AESRD	-	Alberta Environment and Sustainable Resource Development
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 °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
DWSP	-	Drinking Water Safety Plan
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWUDI	-	Groundwater under the direct influence of surface water
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
IFID	-	Intermittent feed and intermittent discharge
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 °C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
USEPA	-	United States Environmental Protection Agency
UV	-	Ultraviolet
WHO	-	World Health Organization

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

- design population of the facility, and
- 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.

Maximum daily design flow (water) - Maximum three 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 4.1.1).

Potable water - As defined in the EPEA. Other domestic purposes in the EPEA definition include water used for personal hygiene, e.g. bathing, showering, washing, etc.

Septage - The liquid, solid or semisolid material removed from septic tanks, portable toilets, and holding tanks that receive sewage from domestic sources. This does not include wastes from grease traps, industrial or commercial processes.

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.

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

Note : All concentrations expressed in milliequivalents per litre

Surface water - Water in a watercourse.

Watercourse - As defined in the EPEA.

4.0 Wastewater Systems - Guidelines

4.1 Design Criteria

4.1.1 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.

4.1.1.1 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:

$$\text{Harmon's Peaking Factor} = 1 + 14/(4 + P^{1/2})$$

where:

P = the design contributing population in thousands.

4.1.1.2 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 4.1 or by actual documented usage.

TABLE 4.1
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

Table 4.1 continued

Place	Estimated Sewage Flow Litres (gallons) Per Day
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 4.1.1.2 (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:

a. Commercial and Institutional Land Uses

The lower limit for Average Flow Generation should be 40 m³/day/ha (0.46 L/s/ha).

b. Industrial Land Uses

The lower limit for average flow generation should be 30 m³/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:

$$Pf = 6.659 (Q_{AVG})^{-0.168}$$

Following from this, the peak dry weather flow rate (Q_{PDW} in L/s) may be determined as follows:

$$Q_{PDW} = \frac{Pf \cdot Q_{AVG}}{6.659 (Q_{AVG})^{0.832}}$$

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 4.1.1.1, and is to be included in the determination of the total generation for the development.

4.1.1.3 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.

4.1.1.4 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.

4.1.2 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.

4.1.3 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.

4.1.4 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.

4.2 Wastewater Collection

4.2.1 Sewers

4.2.1.1 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.

4.2.1.2 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:

$$\text{Required sewer capacity} = \frac{\text{Estimated design flow}}{0.86}$$

Manning equation is generally used in sizing the sewers:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

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).

4.2.1.3 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.

4.2.1.4 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.

4.2.1.5 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 ft³ per minute per ft² of internal surface. Test methods to the requirements of Uni-Bell Standard UNI-B-6-90; see Appendix 1-E for test time calculation.

4.2.2 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.

4.2.2.1 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 snowmelt.

4.2.2.2 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.

4.2.2.3 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.

4.2.2.4 Channelling and Benching

Good design practice should prevent the depth of flow from being above the sidewalls 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.

4.2.2.5 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

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design to eliminate the possibility of a person falling into the manhole if the cover has been dislodged.

4.2.2.6 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 roadway, and where possible, steps should be aligned so that the person exiting from the manhole should do so facing towards oncoming traffic.

4.2.3 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.

4.2.4 Wastewater Pump Station

4.2.4.1 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.

4.2.4.2 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 3.3.2.

a. Equipment Removal

Provision should be made to facilitate removing pumps, monitors, and other mechanical and electrical equipment.

b. 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

a. 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 4.3.1.4. Where a bar rack is provided, a mechanical hoist should also be provided.

b. 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.

c. 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 4.2.4.3.

d. 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.

e. 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.

f. 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.

g. 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

a. Suction Line

Suitable shut-off valves should be placed on the suction line of dry pit pumps.

b. 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

a. 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.

b. 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.

c. Floor Slope

The wet well floor should have a minimum slope of one-to-one 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.

d. 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.

4.2.4.3 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 4.2.4.2. 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.

4.2.4.4 Submersible Pump Stations

Submersible pump stations should meet the applicable requirements under Section 4.2.4.2, 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

a. 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.

b. 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.

c. 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 4.2.4.2 (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 4.2.4.4 (2).

4.2.4.5 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.

4.2.4.6 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 4.2.4.6 (2).

4. Equipment Requirements

a. 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:

i. 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 4.2.4.5. 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.

ii. Size

The engine should have adequate rated power to start and continuously operate under all connected loads.

iii. Fuel Type

Reliability, and ease of starting, especially during cold weather conditions, should be considered in the selection of the type of fuel.

iv. Engine Ventilation

The engine should be located above grade with adequate ventilation of fuel vapours and exhaust gases.

v. 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.

vi. Protection of Equipment

Emergency equipment should be protected from damage at the restoration of regular electrical power.

b. Engine-Driven Pumping Equipment

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

i. 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.

ii. 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 4.2.4.6 (4) (ii) (c).

iii. 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.

c. 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:

i. 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.

ii. 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 4.2.4.6 (4) (iii) (c).

iii. 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.

4.2.5 Forcemains

4.2.5.1 Velocity and Diameter

At design pumping rates, a cleansing velocity of at least 0.6 m/s should be maintained.

4.2.5.2 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.

4.2.5.3 Termination

Forcemains should enter the gravity sewer system at a point not more than 600 mm above the flow line of the receiving manhole.

4.2.5.4 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.

4.2.6 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.

4.3 Wastewater Treatment

4.3.1 Mechanical Wastewater Treatment

4.3.1.1 Site Selection

1. Plant Location

Some of the factors which should be taken into consideration when selecting a new plant site are as follows:

- a. setback distances from land use surrounding plant site [see Section 3.4.3.2 (1)];
- b. susceptibility of site to flooding [See Section 3.4.1.3 (3)];
- c. prevailing wind direction; and
- d. 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:

- a. 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;
- b. 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;

- c. 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
- d. 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.

4.3.1.2 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 4.2.4.2 (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:

- a. head losses due to channel and pipe wall friction;
- b. head losses due to sudden enlargement or sudden contraction in flow cross section;
- c. head losses due to sudden changes in direction, such as at bends, elbows, Y-branches and tees;
- d. head losses due to sudden changes in slope, or drops;
- e. head losses due to obstructions in conduit;
- f. head required to allow flow over weirs, through flumes, orifices and other measuring, controlling, or flow division devices;
- g. head losses caused by flow through comminutors, bar screens, tankage, filters and other treatment units;
- h. head losses caused by air entrainment or air binding;
- i. head losses incurred due to flow splitting along the side of a channel;
- j. head increases caused by pumping; and
- k. 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.

4.3.1.3 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 microorganisms 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:

$$\frac{\text{BOD of the industrial flow at 5 days and } 20^{\circ}\text{C}}{\text{BOD of the industrial flow at 20 days and } 20^{\circ}\text{C}}$$

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 can be tolerated by the micro-organisms. Thus, waste characterization is extremely important in the design of the plant.

4.3.1.4 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

a. Coarse Screens

i. Where Required

Protection for pumps and other equipment should be provided by trash racks, coarse bar racks, or coarse screens.

ii. 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.

b. Fine Screens

i. 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.

ii. 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.

a. Grit Channels

i. Where Required

In advance of pumping or treating units behind screening devices.

ii. 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).

b. Aerated Grit Chambers

i. 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.

4.3.1.5 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

- a. Minimum water depth - 2.1 m
- b. Depth to length ratios (rectangular tank) - 1/10 to 1/30

- c. Surface loading for tanks not receiving return sludge; based on average design flow - $< 0.47 \text{ L/s/m}^2$
- d. Surface loading for tanks not receiving return sludge; based on peak design flow - $0.71 \text{ to } 1.42 \text{ L/s/m}^2$
- e. Surface loading for tanks receiving return sludge; based on peak design flow - $< 0.47 \text{ L/s/m}^2$
- f. Weir overflow rate - $1.74 \text{ to } 5.21 \text{ L/m/s}$
- g. For scum removal, a scum baffle extending at least 150 mm below the surface is necessary close to the overflow weir.

4.3.1.6 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 4.3.1.8.

2. Suspended Growth Systems

a. Continuous-flow activated sludge process

i. 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.

ii. Pre-treatment

The minimum level of pre-treatment 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.

iii. 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.

iv. 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 Cl₂ 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.

v. 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.

vi. 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 4.13 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 re-aeration 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 4.2 TYPICAL ACTIVATED SLUDGE DESIGN PARAMETERS							
Process Modification	Flow Regime	Food to Micro-organism Ratio (g BOD/d) (g MLSS) ^a	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 freeboard 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.

vii. *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 4.2. 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.

viii. 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.

ix. 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.

b. Sequencing Batch Reactors

i. 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.

ii. Pre-treatment

The minimum level of pre-treatment should include grit removal and screening.

iii. 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 multi-reactor 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 pre-reaction compartment at the influent end terminating in a baffle.

iv. 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.

v. 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 Table 4.2. 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*

An 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 freeboard 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.

vi. *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.

vii. 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.

viii. 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.

ix. 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 flow rate because of the intermittent discharge. If the decanting devices are of varying flow rate design, the peak flow rate 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.

x. 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**a. Rotating Biological Contactor****i. General**

The rotating biological contactor (RBC) process may be used where wastewater is amenable to biological treatment.

ii. Pre-treatment

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, pre-aeration should be provided upstream of the RBC process.

iii. 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 m² 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 m² per shaft and should be used only after the second shaft.

iv. 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.

v. Design Loading

The typical design loadings for non-nitrifying rotating biological contactors are shown in Table 4.3.

TABLE 4.3 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

vi. 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.

vii. 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.

viii. Dewatering

The design should provide for dewatering of RBC tanks.

ix. 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.

x. 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.

xi. 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

a. Oxygen Requirements

Secondary biological systems should generally be designed to supply oxygen to satisfy the carbonaceous 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.

i. Carbonaceous Biochemical Oxygen Demand

The carbonaceous oxygen demand may be estimated based on the process oxygen balance using the following formula.

$$R_c = Q (S_o - S)(1 + b O_c - BY)/(1 + b O_c)$$

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 O₂ / kg CBOD)

5	1.0
10	1.1
15	1.2
20+	1.3

ii. 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.

iii. Inorganic Chemical Oxygen Demand

This oxygen demand is most often estimated based on a stoichiometric calculation.

iv. 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.

v. 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.

TABLE 4.4 MIXING REQUIREMENTS

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

b. 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 non-monetary considerations. Under cold climatic conditions, special care should be given to freeze protection if surface aerators are proposed.

i. 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.

Flow meters 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 Fine Pore Diffused Aeration Manual.

ii. 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.

c. 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.

4.3.1.7 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, and (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)

Biological Phosphorus Removal 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 seven (7) percent solids. Phosphorus is removed via wasting sludge. The luxury uptake phenomenon 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, and (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 practiced 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

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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 $\text{NH}_4\text{-N}$ 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 denitrification into the design, approximately two thirds of the oxygen required for nitrification can be recovered.

4. Effluent Disinfection

a. General

i. Methods of Disinfection

Bactericides, Viricides and Potential Disinfectants

UV radiation, chlorine and bromine chloride may be used in full-scale plants to reduce microorganisms (bacteria and viruses) in the wastewater effluent.

Disinfectants

Ozone and chlorine dioxide may be used in full-scale plants to reduce microorganisms (cysts, bacteria, and viruses) in the wastewater effluent.

ii. Selection Method

The selection of a disinfection method should be based on both monetary and non-monetary considerations. In conducting cost comparison,

capital, O&M and total life-cycle costs should be estimated. The non-monetary considerations should include disinfection effectiveness, state of development, effluent quality impacts, chemical hazards and safety concerns, process complexity and ease of operation and maintenance.

b. UV Radiation

i. General

UV radiation may be used to achieve the disinfection requirements to produce effluent that meets Alberta surface water bacteriological 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.

ii. 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.

iii. Selection of UV Disinfection Alternatives

The factors which affect the selection of the appropriate UV system include:

- design flow rate
- wastewater transmittance and suspended solids levels
- effluent bacteria standards

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- 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.

iv. 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 flow rates, 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 should 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 4.5.

TABLE 4.5	
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

v. 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 a 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 flow rates. 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.

vi. 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:

$$\text{Design UV dose} = \text{minimum UV dose} / F_p / F_t$$

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 4.6 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 4.6 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 flow rate. 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.

vii. 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 two 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 two or four 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 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 flow rate signals.

viii. 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 two (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 flow rates during the initial years of operation are significantly lower than the design flow rates, 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.

ix. **Lamp Cleaning**

Adequate facilities should be provided to facilitate cleaning of lamp sleeves. In addition to the cleaning chemical system, appropriate wash down 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.

x. 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.

xi. Ballasts

The ballasts should be compatible with the proposed lamp type. If possible, the more energy efficient electronic ballast should be specified.

xii. 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 by-

products, it will increase the O&M costs necessarily by wasting power and reducing useful lamp life.

xiii. 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.

xiv. 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 flow rates should be decided based on the decant flow rates. If decant system is not designed to have constant-flow design, the maximum flow rate 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.

xv. Safety

Three safety issues should be addressed fully in the design of a UV disinfection facility. They are: (1) exposure to UV radiation, (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.

c. Ozonation

i. 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.

ii. 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.

iii. 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.

iv. 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.

v. 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.

vi. *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 counter current)
- 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 six 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.

vii. 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.

viii. 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.

ix. 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.

x. 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/m³ for eight (8) hours or more per workday, and that no worker be exposed to a ceiling concentration of ozone in excess of 0.6 mg/m³ 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 .

Eyewash 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.

d. Chlorine

i. General

Chlorine is an effective disinfectant. Potential drawbacks to its use include:

- toxicity to aquatic, estuarine and marine organisms
- generation of harmful chlorinated disinfection by-products
- 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.

ii. Forms of Chlorine

Chlorine may be added to the wastewater in the form of liquid / gaseous chlorine or sodium hypochlorite.

iii. 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
- Dechlorination system.

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.

iv. 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 flow rates. A parshall flume is not a satisfactory location for hydraulic mixing.

- v. 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 back mixing. 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.

- vi. 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 re-aeration should be considered. Continuous monitoring of the effluent would be a requirement.

- vii. Chlorination equipment and chlorine room design requirements

For details of chlorine equipment requirements and chlorine room design requirements, see Section 1.5.3.1 (2) and (3).

4.3.1.8 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 4.7.

**TABLE 4.7
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/m².

** Greater water depths are recommended for clarifiers in excess of 372 m² surface area. Less than 3.7 m water depths may be adequate for package plants with average design flow less than 100 m³/d.

4.3.1.9 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 AESRD to confirm the testing requirements.

4.3.1.10 Flow Measurements

1. Location

Flow measurement facilities should be provided to measure the following flows:

- a. plant influent and effluent flow;

- b. plant and process unit bypasses; and
- c. 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.

4.3.1.11 Colour Codes

Refer to Table 4.8 for recommended colour coding for wastewater treatment plant piping.

TABLE 4.8
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

- Entire length of pipe to be painted in basic colour.
- Bands, if required, are to be placed as follows:
 - at 9 m intervals, and / or
 - where the pipe enters and leaves a room.
- Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

4.3.2 Aerated Lagoons

4.3.2.1 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.

4.3.2.2 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:

$$\frac{L_e}{L_i} = \frac{1}{\left[1 + \frac{K_d T}{n}\right]^n}$$

where:

L_e	=	Effluent BOD, mg/L
L_i	=	Influent BOD, mg/L
K_d	=	Reaction rate coefficient at $t^\circ\text{C}$, 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 $t^{\circ}\text{C}$, day^{-1}
- K_{20} = Reaction rate coefficient at 20°C , day^{-1}
- t = Wastewater temperature, $^{\circ}\text{C}$
- 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 $6\text{-}10 \text{ w/m}^3$ 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.

4.3.2.3 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/m³ 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 4.3.2.2 (1) for a single cell, CBOD reduction (complete mix cell) as a % of the average daily design CBOD load (E_1);

$$E_1 = \frac{K_1 T}{1 + K_1 T} \times 100$$

CBOD reduction (aerated cell) as a % of the average daily design CBOD load (E_2);

$$E_2 = \frac{K_2}{1 + K_1 T} \times E_1 \times 100$$

CBOD reduction (overall) $E_3 = E_1 + E_2$

K_1 values can be determined utilizing the formula noted in section 5.3.2.2(2).

Insert the calculated % reduction values on line 1 in the summary Table 5.8.

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 4.8 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 foregoing 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 4.8.

- 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°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 °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 4.9.

**TABLE 4.9
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.

4.3.3 Odour Control

4.3.3.1 Odour Production

1. Odour Development

Wastewater contains numerous potentially odorous 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 four dilutions to reach ED_{50} will contain 5 OU (four dilutions plus the original volume).

The designer must determine, in consultation with the Owner and AESRD (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.

4.3.3.2 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.

4.3.3.3 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.

a. 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 modelling of sulphide generation.

b. Gas Phase Analyses

In-situ gas phase testing can be used to identify a wide range of odorants, 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.

c. 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.

d. Gas Chromatography

GC analyses are useful for identifying total levels of sulphides, and other potential odorants. 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 Modelling

a. 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.

b. Air Quality Computer Modelling

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

4.3.3.4 Odour Control and Abatement Measures

It is generally more reliable and cost effective to treat and remove odorants 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.

a. 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.

b. 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.

c. 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.

d. 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:

- a. Minimize elevation differences where streams converge;
- b. Introduce side streams below the liquid surface;
- c. Use of submerged effluent weirs and downstream flow control in lieu of conventional launders for sedimentation tanks and clarifiers;
- d. Avoid excessive or unnecessary aeration;
- e. Avoid the use of screw lift pumps on potentially odorous 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.

a. 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.

b. 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.

c. 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:

- i. chemical scrubbers, packed bed or mist contactor types;
- ii. activated carbon, with or without chemical impregnation;
- iii. activated alumina impregnated with potassium permanganate;
- iv. biofilter, in-vessel or soil / compost types;
- v. incineration;
- vi. 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.

4.3.4 Instrumentation and Controls

4.3.4.1 General

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. Alberta Environment and Sustainable Resource Development 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.

4.3.4.2 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

a. Magnetic Flow meters (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.

b. Ultrasonic Flow meters

Flow meter Construction. The flow meter 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 is not recommended for influent wastewater, primary sludge, thickened sludge, nitrification RAS, or nitrification WAS. Reflective style is not recommended for primary effluent, secondary clarifier effluent final effluent or process wash water.

c. Turbine Flow meters

Flow meter Construction. The flow meter 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.

d. 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. They 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

a. 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.

b. 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.

c. 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

a. 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.

b. 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.

c. 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

a. 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.

b. 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.

c. 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.

d. Thermal Bulb

Installation. No special installation requirements.

Applications. Thermal bulbs are suitable for temperature measurement applications with ranges of 0-500 C.

4.3.4.3 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.

a. Level Control

Lift stations vary in size and storage capacity but generally they require similar controls. The level in the wet well 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 wet well. 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.

b. 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 4.2.4.2 (5)].

c. 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.

d. 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.

e. Alarms

Lift stations should be alarmed as outlined in Section 4.2.4.5.

2. Mechanical Bar Screens

Three methods are used to control the operation of mechanical bar screens:

- a. Simple manual start / stop which requires the presence of the operator at the screen in order to start and stop the screen.
- b. 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 (i.e. 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.
- c. 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 (i.e. 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

a. 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.

b. 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

a. 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:

i. Closed Loop Control (Feedback Control)

Closed loop control consists of on line dissolved oxygen analyzers providing feedback control to an airflow 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 airflow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

ii. 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 feedback to the airflow control elements.

Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, airflow control elements.

b. 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.

c. 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.

d. 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.

e. 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.

a. 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.

b. 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 their capabilities and performance. The selection of hardware and software should be done carefully to assure current performance and future supportability and expendability.

4.3.4.4 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.

4.3.4.5 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.

4.3.4.6 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.

4.3.5 Emergency Facilities and Component Reliability

4.3.5.1 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 backup 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.

4.3.5.2 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.

4.3.5.3 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 (four 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, continuous disinfection should be provided during all power outages.

4.4 Residual / Biosolids (Sludge) Processing

4.4.1 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

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6. methods of ultimate disposal.

4.4.2 Biosolids (Sludge) Handling and Treatment

4.4.2.1 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 given 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/m³ or mechanical means - 6.6 W/m³). 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/m³/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/m³ 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/m³ 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 valves 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 CBOD₅ 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*

a. *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.

b. *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.

c. *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/m³.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/m³.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 common wall construction or earthen-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.

4.4.2.2 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.

4.4.2.3 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 5.10. 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 4.9 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/m².d; waste activated 12 to 36 kg/m².d; combination of primary and waste activated based on weighted average of above loading rates.
- *Overflow Rate*
0.19 to 0.38 L/m².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 4.10
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/m².d (without flocculating chemicals); up to 240 kg/m².d (with flocculating chemicals).
- *Chemical Conditioning*
Feed chemical to mixing zone of sludge and recycled flow.
- *Hydraulic Feed*
Up to 1.74 L/m².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.

4.4.2.4 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 4.11 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 4.11 assume that the sludges have been properly conditioned.

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 AESRD before proceeding with the design of the facility.

**TABLE 4.11
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%)	360	Yes	No
		Digested Primary + WAS- 35 to 50%)		Yes	No
		WAS- (25 to 50%)		Yes	No
Centrifuge (Solid Bowl)	95 to 99	Raw or Digested Primary + WAS - (15 to 25%)	360	Yes	No
		WAS - (12 to 15%)		Yes	No
Belt Filter	85 to 95	Raw or Digested Primary + WAS - (14 to 25%)	130	Yes	No
		WAS - (10 to 15%)		Yes	No

4.5 Septage Management

Municipal wastewater facility and the independent septage management facility owners accepting hauled wastewater or septage should follow guidelines outlined in this section and may outline additional requirements as they see appropriate to manage their wastewater system.

4.5.1 Treatment and Disposal of Septage

Septage consists of higher concentration of solids and organics, as well it may contain varying levels of oil and grease and inorganic, making it difficult to handle and treat. The management, treatment and disposal of septage are affected by its chemical and physical characteristics. In addition the method of treatment and disposal must also consider the site specific needs to minimize odour issues, address public health and minimize environmental risks.

Many different factors affect the physical properties of septage including septic tank size, user habits, and pump-out frequency. Therefore a good knowledge of septage characteristics and variability is important in determining appropriate treatment or disposal methods.

An engineering evaluation of the wastewater treatment facility may be required when the volume of the septage received by the wastewater facility exceeds 5% of the wastewater treatment facilities' capacity for instantaneous flow rate or mass loading of solids, metal or other parameter. The engineering evaluation should include process impact analysis through process modeling, mass balances or another approved method.

4.5.2 Septage Management Systems

Septage receiving system including the receiving facility, pipes, holding chamber and pumps should be designed for a minimum of 10 years design flow with provision for expansion to handle 20 or 25-year design flow.

The design of the septage receiving facility at the wastewater treatment plant or at an independent septage management facility should include:

1. Capability to control access to the facility;
2. A hard surface unloading ramp sloped to a drain to collect spillage. The ramp drainage must discharge to the treatment facility and must avoid or limit collection of storm water.
3. Capability to monitor and track septage haulers discharging to the facility;
4. Capability to collect a manifest from each septage hauler;
5. Automated sampling systems to collect a representative sample of truck load discharging at the facility as well as composite sample of loads over a 24-hour period;
6. Odour management system;
7. Flow equalization system;
8. A flexible hose fitted with easy connect coupling to provide for direct connection from the hauling truck outlet to minimize spillage;
9. Heating of discharge chamber and / or receiving piping;
10. Metering and billing systems to monitor septage received and provide accurate billing information to septage haulers and operator(s) of the wastewater treatment facility;
11. Wash down system to clean the septage receiving station;
12. Rock traps, grinding, screening, grit, and / or grease removal where appropriate to protect the downstream treatment units;
13. Valving, piping and short storage for operational flexibility to allow the control of the flow rate and point of septage discharge to the wastewater system; and
14. Provide capability for visual inspections for floating grease, petroleum, oily sheen, visible solids, and colours

In addition, precautionary measures are recommended to control excessive generation of hydrogen sulphide to prevent odours, damage to equipment and hazardous conditions for operators. Chemical or physical treatment including ventilation may be necessary.

4.5.3 Monitoring and Record Keeping for Septage Management

A manifest should be prepared for each hauled load and that this information is collected by the owners of the septage management system. The manifests should include the following information:

1. Name and addresses of location(s) where the septage was collected;
2. Volume and type of load (e.g. holding tank / septic tank) of septage collected from each location;
3. Date septage collected;
4. Name of septage receiving facility discharged to;
5. Date and time when septage was disposed to the receiving facility; and
6. Name and address of septage hauling company.

A septage sampling program should be developed by the owner of the septage management system to obtain representative characteristics of the hauled wastes. Samples should be tested for BOD, COD, TSS, TP, pH, oil and grease, metals, and organics.

The owner of the septage management system prepare summary of the monthly report of the manifests and the sampled collected. A copy of this information should be kept for a minimum period of 5 years. The monthly summary should include the following:

1. Daily septage volume received;
2. Monthly septage volume received from each hauler and total monthly volume;
3. Daily volume of septage diverted to the treatment process (if different from (1.);
4. Results of inspection and random sampling carried out during the month; and
5. Other observations where appropriate.

4.5.4 Criteria for Land Application of Septage

The disposer of septage to land must obtain a Letter of Authorization from Alberta Environment and Sustainable Resource Development prior to land application of septage.

The disposer of septage to land is defined as the person named in a letter of authorization from Alberta Environment and Sustainable Resource Development being authorized to dispose of septage to a land. The disposer shall haul all septic or holding tank waste in a contained vessel and shall only dispose of septic or holding tank waste if the septic tank or holding tank receives only domestic wastewater.

The disposer shall only apply septage to land subject to the following provisions:

1. The disposer shall not apply septage when ice, snow or frozen conditions exist;
2. The disposer shall not apply septage to dry stream and dry intermittent drainage areas; and
3. The disposer shall only apply septage to land that meets the requirements identified in Table 4.12

**TABLE 4.12
SITE SUITABILITY**

Characteristic	Type of site			
	Casual application site*		Designated application site	
	Acceptable	Unacceptable	Acceptable	Unacceptable
Soil PH	-	-	>6.5	<6.5
Soil Texture	All texture except sand & gravel	Sand & gravel	All texture except sand & gravel	Sand & gravel
Slope of the land	<5%	>5%	<5%	>5%
Depth to groundwater aquifer	-	-	>2m	<2m

* Casual application site is considered as a site where septage is applied less frequent than once every five years.

4.5.4.1 Separation Distances from Specified Features

The disposer shall apply septage in accordance with the separation distances from specified features set out in Table 4.13.

**TABLE 4.13
MINIMUM SEPARATION DISTANCES FROM SPECIFIED FEATURES**

Feature	Minimum Distance (in metres)	Preferred Distance (in metres)
Rivers, Canals, Creeks, Intermittent Drainage Courses, Lakes, Sloughs, Dugouts	30	50
Water Wells	50	50
Areas Zoned Residential or Devoted to Urban Use	500	800
Occupied dwellings	60	100
Road Allowances	10	20
Public Building Perimeter	10	30
Public Buildings	60	100
School Yard Boundaries	200	500
Cemeteries, Playgrounds, Parks, Campgrounds	200	500
Property Boundary	60	100

4.5.4.2 Methods of Application

The disposer shall only apply septage by injection or surface application.

If surface application is employed, the disposer shall till the land within 48 hours of the surface application to incorporate the stabilized septage with the surface soil material.

4.5.4.3 Application Rates

The disposer shall not exceed the single and annual application rates for casual and designated application sites as specified in Table 4.14.

TABLE 4.14
MAXIMUM APPLICATION RATES FOR CASUAL & DESIGNATED APPLICATION SITES

Waste type	Maximum Single and Annual Application Rates			
	Casual Application Rates*		Designated Application Rates	
	Single application	Annual application	Single application	Annual application
Septic Tank	100 m ³ /ha	Not allowed	100 m ³ /ha	500 m ³ /ha
Holding Tank	100 m ³ /ha	Not allowed	100 m ³ /ha	300 m ³ /ha

* Casual applications rates are rates applied on Casual application site which are considered as sites where septage is applied less frequently than once every five years.

4.5.4.4 Restrictions on Land Use After Application

The disposer shall only apply septage to land intended for the production of forages, oil seeds, small grains, trees and commercial sod.

The disposer shall not apply septage to land that is intended to be used for the production of root crops, vegetable and fruit crops, or dairy farming pasturing within three years of the application of the septage.

4.5.4.5 Requirement for Written Permission from Landowner

The disposer shall obtain written permission from the landowner prior to the application of septage.

4.5.4.6 Record Keeping

The disposer shall maintain a minimum of the following records:

1. Address of all clients;
2. Volume of septage collected from each client;
3. Land location of disposal of the septage collected;
4. Method and application rate of the septage disposed; and
5. Date of application of the septage and date of till if surface application was employed.

All records shall be kept by the disposer for a minimum of five (5) years from the date of the septage application.

4.6 Wastewater Systems Operation and Monitoring

4.6.1 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.

4.6.2 Reliability

1. The wastewater system must 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.

4.6.3 Operations

1. The wastewater systems must be managed and operated in accordance with the EPEA approval of the systems. The wastewater treatment facilities must 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:
 - a. General procedures for routine or major emergencies within the wastewater system; and
 - b. 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 AESRD when results of an inspection conducted by AESRD or monthly returns indicate conditions which are currently or may become a detriment to system operations.

4.6.4 Facility Classification and Operator Requirements

4.6.4.1 Facility Classification

On recommendation from Water and Wastewater Operators Certification Advisory Committee, AESRD 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 4.15(a) and 4.15(b).

Table 4.16 summarizes the classification system. The classification system is based on the "degree of difficulty to operate" a facility. The Alberta system is similar to those used across Canada and the United States.

4.6.4.2 Requirement For Having Certified Operators

In accordance with EPEA, day-to-day operations of wastewater systems must 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.

4.6.4.3 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 ensure 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 (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

Certified operators along with the Approval or Registration holder 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 (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

4.6.4.4 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 AESRD 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 4.15(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) (Population Served _____)	(0 – 5)
Design flow (avg. day) or peak month's flow (avg. day), whichever is larger (1 pt/5000 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	(8)
Dissolved air flotation (not sludge)	(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, electro dialysis, membranes, cloth filters	(15)
Chemical recovery, carbon regeneration	(4)
Solids Handling	
Conditioning	(2)
Thickening	(5)
Anaerobic digestion	(10)
Aerobic digestion	(6)
Evaporation sludge drying	(2)
Mechanical dewatering	(8)
Solids reduction (incineration, wet oxidation)	(12)
Composting (on site)	(8)

Table 4.15(a) continued

Disinfection	
Chlorination or comparable On-site generation	(5) (5)
Use of SCADA (see note)	(0 – 6)*
Laboratory Control by Plant Personnel	
Bacteriological (complexity)* Chemical / Physical (complexity)*	(0 – 10)* (0 – 10)*

* See Table 4.15(b)

TABLE 4.15(b)
WASTEWATER TREATMENT PLANT CLASSIFICATION POINT GUIDE

Note: Each unit process should have points assigned only once

ITEM	POINTS
EFFLUENT DISCHARGE	0 - 6
Receiving Stream Sensitivity The Key concept is the degree of dilution provided under low flow conditions Suggested point values are:	
No discharge	0
Secondary treatment is adequate	1
More than secondary 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 (slight to extreme)	0 - 6
The key concept is frequency and / or intensity of deviation or excessive 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	0 - 10
Bacteriological / biological (complexity) The Key concept is to credit bacti / bio lab work done on-site by plant personnel	
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

Table 4.15(b) continued

Chemical / physical (complexity)	0 – 10
The key concept is to credit chemical / physical lab work done on-site by plant personnel	
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.	Up to 7
Highly sophisticated instrumentation such as atomic absorption and gas chromatography	10

USE OF SCADA	0 - 6
Provide data with no process adjustment	0
Provide data with limited process adjustment	2
Provide data with moderate process adjustment	4
Provide data with extensive or total process adjustment	6

TABLE 4.16
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 4.12(a))	30 or fewer	31-55	56-75	76 or more

Notes: AESRD 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.

4.6.4.5 Facilities Staffing Requirements: Certified Operators

For **Class I Wastewater Treatment, and Wastewater Collection facilities**, there must be a certified Level I (or higher) operator in charge of the day-to-day operation of that facility. The exception to this is when the approval or registration states that an operator with a Small Systems certificate is acceptable. A back-up certified operator is recommended.

For **Class II Wastewater Treatment and Wastewater Collection 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 Wastewater Treatment 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. It is further recommended that other operators be certified.

For **Class III Wastewater Treatment serving a population of 1,501 – 15,000**, 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. It is further recommended that other operators be certified.

For Class III Wastewater Treatment facilities serving a population of 15,001 – 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. It is recommended that other operators be certified.

For Class III Wastewater Collection systems serving a population of 15,001 – 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. It is recommended that other operators be certified.

For Class III Wastewater Treatment 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. It is recommended that other operators be certified.

For Class IV Wastewater Treatment 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. It is recommended that other operators be certified.

For Class IV Wastewater Collection systems serving a population of 50,001 or more, 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. It is recommended that other operators be certified.

For Class IV Wastewater Treatment 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. It is recommended that other operators be certified.

4.7 System Monitoring

4.7.1 General

Alberta Environment and Sustainable Resource Development 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.

4.7.1.1 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, AESRD would require two types of samples to be collected, "composite" or "grab."

"Composite sample" means a sample consisting of not less than twenty-four discrete portions of equal volume collected as follows:

- a. 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 24 hours, or
- b. sequentially at regular time intervals over a period of 24 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:

- a. 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 AESRD; or
- b. Collection, preservation and the analysis of samples are performed by a laboratory approved by AESRD.

4.7.1.2 Approval of Analytical Procedures

Owners should ensure that laboratories obtain approval from AESRD for the use of any analytical procedures not included in the "Standard methods."

The laboratory would be required to follow a protocol established by AESRD for the approval of analytical procedures not included in the "Standard Methods."

4.7.2 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. 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.

4.7.2.1 Activated Sludge Systems

Table 4.17 lists some of the significant parameters that should be monitored to ensure proper operation of an activated sludge system.

**TABLE 4.17
OPERATIONAL MONITORING**

<u>Parameters</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Monitoring Frequency</u>
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
Peak flow (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

4.7.2.2 Wastewater Lagoons

Table 4.18 outlines the operational monitoring requirements for wastewater stabilization ponds.

**TABLE 4.18
OPERATIONAL MONITORING**

Requirements	Monitoring Frequency
Actual dates and duration of discharge	Annually for every discharge
Volume of discharge	Annually for every discharge
Inspection of the discharge route	Prior each discharge the discharge route should be inspected to identify changes / site specific concerns and modify discharge practice as necessary. Issues such as stream flows, other discharges either upstream or down stream, stream blockage, etc... should be documented and addressed as necessary.
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 AESRD.	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.

4.7.3 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 AESRD. 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.

4.7.3.1 Secondary Treatment - Mechanical (for current population < 20,000)

1. Continuous Discharge to a body of water

TABLE 4.19
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.20
TREATMENT PERFORMANCE AT COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.21
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING**

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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.

4.7.3.2 Tertiary Treatment - Mechanical (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 4.22
TREATMENT PERFORMANCE AND COMPLIANCE

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.23
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.23 - Continued

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.21.

4.7.3.3 Aerated Lagoons (for current population < 20,000)

1. Continuous discharge to a body of water

TABLE 4.24
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.25
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.21.

4.7.3.4 Aerated Lagoons (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 4.26
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Parameter</u>	<u>Point of Measurement</u>	<u>Requirement</u>	<u>Type of Sample</u>	<u>Minimum Monitoring Frequency</u>
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 4.27
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.

4.7.3.5 Wastewater Lagoons

TABLE 4.28
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

<u>Type</u>	<u>Minimum Requirements</u>
Wastewater stabilization pond built to the specified design configuration as per Section 3.4.1.2.	Effluent quality standard not specified Discharge from the storage cell once a year between late spring and fall. Early spring discharge allowed under exceptional circumstances Prior each discharge the discharge route should be inspected to identify changes / site specific concerns and modify discharge practice as necessary. Issues such as stream flows, other discharges either upstream or down stream, stream blockage, etc... should be documented and addressed as necessary. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

4.7.4 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: Wastewater Systems – Performance and Design Standards. Issue oriented monitoring and follow-up actions for various incidents are outlined in Table 4.29.
2. When a violation of the prescribed effluent standard occurs, the owner should:
 - a. notify AESRD in accordance with Section 4.9;
 - b. determine the cause of the problem; and
 - c. take action as directed by AESRD.

4.7.4.1 Activated Sludge System

Issue oriented monitoring and follow-up actions for various incidents in Activated Sludge Systems are outlined in Table 4.29.

TABLE 4.29
ISSUE ORIENTED AND FOLLOW-UP MONITORING

<u>Incident</u>	<u>Parameter</u>	<u>Point of Measurement</u>	<u>Type of Sample</u>	<u>Monitoring Frequency</u>	<u>Follow-Up</u>
Plant pass	by-flow	By-pass line	N/A	Each incident	-
	CBOD		Grab/composite		
	TSS		Grab/composite		
	Coliform (total)		Grab		
	Coliform (fecal)		Grab		
	NH ₃		Grab/composite		
Sludge bulking	Total P		Grab/composite		
	CBOD (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		
Rising Sludge/ Denitrification	NO ₃ -N	RAS line	Grab		
	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		
	Flow	Primary sludge line	N/A		
Primary Sludge Septicity	Retention time			For the duration of the event	Increase sludge wasting, monitor retention time
	Density				
	CBOD (raw)				
	TSS (raw)				

4.8 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 AESRD and shall send the records to AESRD 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:
 - a. the date, place and time of sampling, and the name of the person collecting the sample;
 - b. identification of the sample type (compliance sample, operational sample, special purpose sample);
 - c. date of analysis;
 - d. laboratory and person responsible for performing analysis;
 - e. the analytical method used; and
 - f. 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.

4.9 Reporting

1. Reporting requirements shall be in accordance with the operating approval for the facility, issued by AESRD.
2. The owner shall report to AESRD immediately upon finding:
 - a. the violation of prescribed effluent standards for that facility; and
 - b. 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 AESRD within one week and remedial action carried out as per EPEA Division 1, Sections 110, 111 and 112 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 AESRD 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:
- a. a monthly summary of all operational and compliance monitoring for that particular facility, as identified by AESRD;
 - b. a summary of approval contraventions and remedial measures taken; and
 - c. a summary of any permanent upgrading works undertaken during the year.

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

Part 5 Stormwater Management Guidelines
of a Total of 5 Parts

March 2013

Part 5 STORMWATER MANAGEMENT GUIDELINES
March 2013

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Additional Parts published separately are:

Part 1 Standards for Municipal Waterworks

Part 2 Guidelines for Municipal Waterworks

Part 3 Wastewater Systems Standards for Performance and Design

Part 4 Wastewater System Guidelines for Design, Operating and Monitoring

FOREWORD TO PART 5 STORMWATER MANAGEMENT GUIDELINES (2013)

Alberta Environment and Sustainable Resource Development (AESRD) has the regulatory mandate, in accordance with the Environmental Protection and Enhancement Act and Regulations, for the Drinking Water, Wastewater and Storm Drainage serving large public systems in Alberta. AESRD 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. AESRD's objective is to develop comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable.

Since publication of the last revision of the Standards and Guidelines, Alberta Environment and Sustainable Resource Development has embarked on a process of "decoupling" the various components of the January 2006 document into functionally-associated sections to aid those using the document. This process started with the publication of the January 2006 version of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in the Alberta Gazette. A program of separating the component parts of this document is under way and new parts will eventually replace the corresponding sections in the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems. Until the process of "decoupling" is completed with new "Parts" the existing sections of the 2006 Standards and Guidelines document will remain in operation. This Part (Part 5) details system components that are guidance to best practices in providing well designed and managed Storm Drainage System.

Engineering consultants and / or the system owners / utilities are responsible for the detailed project design and satisfactory construction and operation of the Storm Drainage systems.

In accordance with the Wastewater and Storm Drainage Regulation (119/1993) storm drainage will be designed so that it meets, as a minimum, the applicable standards set out in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AESRD, as amended or replaced from time to time, or, any other standards and design requirements specified by the Regional Director.

AESRD last revised its Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in January 2006.

This present part is intended to provide general guidance on for storm drainage management. Good engineering and best management practices are included in this Part. These are not mandatory requirements but they establish the minimum expectation when the system owner / utility applies for registration.

The only change from the January 2006 version of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems is the numbering of Section 6 – Stormwater Management Guidelines. This document, Part 5 – Stormwater Management Guidelines is now numbered 5.0 through 5.3.6.4.

DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AESRD	-	Alberta Environment and Sustainable Resource Development
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 °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
DWSP	-	Drinking Water Safety Plan
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWUDI	-	Groundwater under the direct influence of surface water
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
IFID	-	Intermittent feed and intermittent discharge
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 °C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
USEPA	-	United States Environmental Protection Agency
UV	-	Ultraviolet
WHO	-	World Health Organization

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

- design population of the facility, and
- 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.

Maximum daily design flow (water) - Maximum three 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 4.1.1).

Potable water - As defined in the EPEA. Other domestic purposes in the EPEA definition include water used for personal hygiene, e.g. bathing, showering, washing, etc.

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.

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

Note : All concentrations expressed in milliequivalents per litre

Surface water - Water in a watercourse.

Watercourse - As defined in the EPEA.

5.0 Stormwater Management Guidelines

5.1 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 AESRD publication entitled, Stormwater Management Guidelines for the Province of Alberta.

5.2 Stormwater Collection

5.2.1 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.

5.2.1.1 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.

5.2.2 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.

5.2.2.1 Sewer Hydraulics

Storm sewer pipe shall be designed to convey the design flow when flowing full with the hydraulic grade-line at the pipe crown. Crown elevations should match at manhole junctions.

5.2.2.2 Flow Velocities and Minimum Slope

Storm sewer flow velocities shall not be less than 0.60 m/s when flowing full. Refer to Table 5.1 for minimum slopes of gravity storm sewers.

If sewer flow velocities exceed 3 m/s, special consideration shall be given to prevent scouring.

TABLE 5.1
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

5.2.2.3 Pipe Size

The minimum diameter for storm sewers shall be 300 mm. The minimum diameter for catch basin leads shall be 250 mm.

5.2.2.4 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).

5.2.2.5 Pipe Cover

The minimum depth of cover to pipe crown shall be 1.2 m.

5.2.2.6 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.

5.2.2.7 Change in Flow Direction

For storm sewer pipes greater than 600 mm in diameter, changes in flow direction at manholes should not exceed 45°. This limitation may be exceeded if care is taken to design a proper transition manhole.

5.2.2.8 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.

5.2.2.9 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.

5.2.3 Storm Manholes

The design of storm manholes should conform in all respects to Section 4.2.2 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.

5.2.4 Catch Basins and Gutters

5.2.4.1 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.

5.2.4.2 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.

5.2.4.3 Catch Basin Construction

All catch basin bodies shall be of either 600 mm or 900 mm pre-cast 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.

5.2.4.4 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%.

5.2.5 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 Regional Director of Alberta Environment and Sustainable Resource Development before commencing with detailed design.

5.3 Stormwater Best Management Practices (BMPs)

5.3.1 Introduction

5.3.1.1 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.

5.3.1.2 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 BMPs.

5.3.2 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, catch basin cleaning, and animal litter removal through enforcement of bylaws.

5.3.3 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 catch basin restrictors or orifices in the storm sewer to promote parking lot detention
- over sizing storm sewers and installing orifices in the sewer to create pipe storage
- installing catch basin restrictors in rear yard catch basins 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 base flow 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 soak away pits
- sump pumping foundation drains to rear yard ponding areas.

5.3.3.1 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 two 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 two percent or higher. Areas outside of this envelope should be graded at less than two percent.

Reduced lot grading BMPs promote depression storage and natural infiltration and reduce risks associated with flooding and erosion. The maintenance of natural infiltration could have positive impacts on base flow 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 re-grading 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 base flows.

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 5.1. Grades within 4 m of structures should be maintained at two 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.

5.3.3.2 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 re-grading 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 base flows.

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 5.2. Maximum depths should be maintained at 100 mm. Flow paths should be provided to direct overland flow to the pond. To maintain the pond, catch basins 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.

5.3.3.3 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 catch basin 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 homeowners. 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 base flows.

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 5.3 and 5.4 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 pre-treatment filter (Figure 5.3) or sump (Figure 5.4) 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.

5.3.3.4 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 / soak away 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 catch basin 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 base flow 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 5.5. 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.

5.3.4 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 catch basins
- grassed swales.

5.3.4.1 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 pre-treatment components. Roadway runoff is normally directed toward grassed areas that act as sediment filters prior to flowing into the stormwater catch basin. The stormwater catch basin is raised to allow some ponding and further sediment removal. The catch basin 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 5.6. Design considerations must include the pre-treatment of road runoff for solids removal. Pre-treatment can be accomplished by incorporating grassed boulevards as pre-treatment 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.

5.3.4.2 Pervious Catch basins

1. Purpose

Pervious catch basins are intended to convey and infiltrate road drainage.

2. Description

Pervious catch basins are normal catch basins with larger sumps that are physically connected to an exfiltration storage media. The storage media is generally located beneath or beside the catch basin.

3. Applicability

Pervious catch basins are still considered to be experimental.

4. Effectiveness

Maintenance requirements for pervious catch basins are dependent on the clogging frequency of the infiltration media which can be high given the sediment load normally associated with road runoff. Pervious catch basins 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 catch basins 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 catch basin for road runoff control is illustrated in Figure 5.7. The most important design consideration is the provision of adequate pre-treatment 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.

5.3.4.3 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 mosquitoes 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 5.8. An illustration of a grassed swale with a check dam is shown in Figure 5.9.

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 5.8 and 5.9, are normally used when the longitudinal slope exceeds 2 to 4 percent. Figure 5.8 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 5.9) 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.

5.3.5 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.

5.3.5.1 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 hydrograph, 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 re-suspension
- permanent pool minimizes blockage of outlet
- promotes biological removal of pollutants

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- 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 5.10 illustrates some of the basic requirements for a wet pond. Detailed design 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 side slopes above active storage zone are 4:1 to 5:1
- maximum interior side slopes in active storage zone are 5:1 to 7:1
- maximum exterior side slopes 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 pre-treatment 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.

5.3.5.2 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 hyetograph, 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 draw down time. The removal of soluble pollutants does not generally occur in a dry pond. Without a permanent pool, re-suspension 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 5.11 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 side slopes of 4:1 to 5:1
- maximum exterior side slopes 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.

5.3.5.3 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 flood fringe 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 wetland's 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 bird watching, 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, wetlands typically flatten and spread the inflow hyetograph, 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 / sub watershed 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 draw down 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 pre-treatment 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 pre-treatment 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 5.12 illustrates the major components of a constructed wetland.

General design considerations are:

- wetland size should be approximately five (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

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- 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), dragonflies, 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, re-suspend 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.

5.3.5.4 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 pre-treatment 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 base flows.

6. Water Quality

Pre-treatment 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 5.13 and 5.14, 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 than 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 48-hour draw down 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.

5.3.5.5 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 aboveground 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 pre-treatment 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 5.15. 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.

5.3.5.6 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 5.16. 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.

5.3.5.7 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 filter is collected by a pervious 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 known about their performance in winter or freshet conditions.

7. Design Considerations

A sand filter application is illustrated in Figure 5.17. 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.

5.3.5.8 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 belowground, 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 (<5 ha) 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 and 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 favourable 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 5.2 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 re-suspension 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 5.18 and 5.19 for illustrations of the two types of oil / grit separators.

5.3.6 BMP Screening and Selection

5.3.6.1 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 5.2 summarizes the advantages and disadvantages of a number of BMPs.

TABLE 5.2 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 re-suspension 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 permeability's - Usually requires pre-treatment 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 permeability's - Pre-treatment 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 5.2 continued

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 runoff.	- Relatively high capital costs compared to manholes - Applicable for drainage areas less than 5 ha.

5.3.6.2 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 5.3 summarizes physical constraints associated with various BMP types.

TABLE 5.3 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 catch basins	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

5.3.6.3 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 BMPs in regard to the objectives for stormwater management are shown in Table 5.4.

TABLE 5.4 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 catch basins	♦*	♦	♦	•
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				

5.3.6.4 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 pre-screened list that will meet the water quality objectives.

The reported effectiveness of a number of BMPs to remove pollutants are shown in Table 5.5.

TABLE 5.5 EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS									
Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Infiltration Basin	Average:	75	65	80	65	65	65	Soil percolation rates	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodward-Clyde, 1966
	Reported Range:							Basin surface area	
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100	Storage volume	
	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	NVPDC, 1979; EPA, 1977; Schueler, 1967; Griffin et al, 1980; EPA, 1963; Woodward-Clyde, 1966; Kuo et al 1968; Lugbill, 1990
	Reported Range:	45-100	40-100	(110)-100	45-100	45-100	45-100	Trench surface area	
	Probable Range:							Storage volume	
	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 5.5 continued EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS									
Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Vegetated Filter Strip	Average:	65	40	40	40	45	60	Runoff volume	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	Slope	
	Probable Range:	40-90	30-80	20-60	-	30-80	20-50	Soil infiltration rates	
	No. of Values Considered:	7	4	3	2	3	3	Vegetative cover Buffer length	
Grass Swale	Average:	60	20	10	25	70	60	Runoff volume	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	Slope	
	Probable Range:	20-40	20-40	10-30	-	10-20	10-20	Soil infiltration rates	
	No. of Values Considered	10	8	4	1	10	7	Vegetative cover Swale length Swale geometry	

TABLE 5.5 continued EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS									
Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Porous Pavement	Average:	35	5	20	5	15	5	Maintenance	Pitt, 1965 Field, 1985 Schueler, 1967
	Reported Range:	0-95	5-10	5-55	5-10	10-25	5-10	Sedimentation storage volume	
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
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 / Filtration Basin	Average:	80	50	35	55	60	65	Treatment volume	City of Austin, 1986 Environmental and Conservation Service Department, 1990
	Reported Range:	60-95	0-90	20-40	45-70	30-90	50-80	Filtration media	
	Probable Range:	60-90	0-80	20-40	40-70	40-80	40-80		
	No. of Values Considered:	10	6	7	3	5	5		

5-35

TABLE 5.5 continued EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS									
Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Water Quality Inlet	Average:	35	5	20	5	15	5	Maintenance	Pitt, 1965 Field, 1965 Schueler, 1967
	Reported Values:	0-95	5-10	5-55	5-10	10-25	5-10	Sedimentation storage volume	
	Probable Values:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
Water Quality Inlet with Sand Filter	Average:	80	NA	35	55	80	65	Sedimentation storage volume	Shaver, 1991
	Reported Range:	75-85	NA	30-45	45-70	70-90	50-80	Depth of media	
	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	Pitt, 1965 Schueler, 1967
	Reported Range:	0-25	5-10	5-10	5-10	10-25	5-10	Outlet configurations	
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	2	1	1	1	1	1		

TABLE 5.5 continued EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS								
Management Practice		Removal Efficiency (%)						References
		TSS	TP	TN	COD	Pb	Zn	
Extended-Detention Dry Pond	Average:	45	25	30	20	50	20	Storage volume
	Reported Range:	5-90	10-55	20-60	0-40	25-65	(-40)-65	Detention time
	Probable Range:	70-90	10-60	20-60	30-40	20-60	40-60	Pond shape
	No. of Values Considered:	6	6	4	5	4	5	
Wet Pond	Average:	60	45	35	40	75	80	Pond volume
	Reported Range:	(-30)-91	10-85	5-85	5-90	10-85	10-95	Pond shape
	Probable Range:	50-90	20-90	10-90	10-90	10-95	20-95	
	No. of Values Considered:	18	18	9	7	13	13	

TABLE 5.5 continued EFFECTIVENESS OF BEST MANAGEMENT PRACTICES FOR CONTROL OF RUNOFF FROM NEWLY DEVELOPED AREAS									
Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Extended-Detention Wet Pond	Average:	80	65	55	NA	40	20	Pond volume	Ontario Ministry of the Environment, 1991 cited in Schueler et al 1992
	Reported Range:	50-100	50-60	55	NA	40	20	Pond shape	
	Probable Range:	50-95	50-90	10-90	10-90	10-95	20-95	Detention time	
	No. of Values Considered:	3	3	1	0	1	1		
Constructed Stormwater Wetlands	Average:	65	25	20	50	65	35	Storage volume	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 Rainelt et al, 1990 cited in Woodward and Clyde, 1991
	Reported Range:	(-20)-100	(-120)-100	(-15)-40	20-80	30-95	(-30)-60	Detention time	
	Probable Range:	50-90	(-5)-80	0-40	-	30-95	-	Pool shape Wetlands biota	
	No. of Values Considered:	23	24	6	2	10	8	Seasonal variation	

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

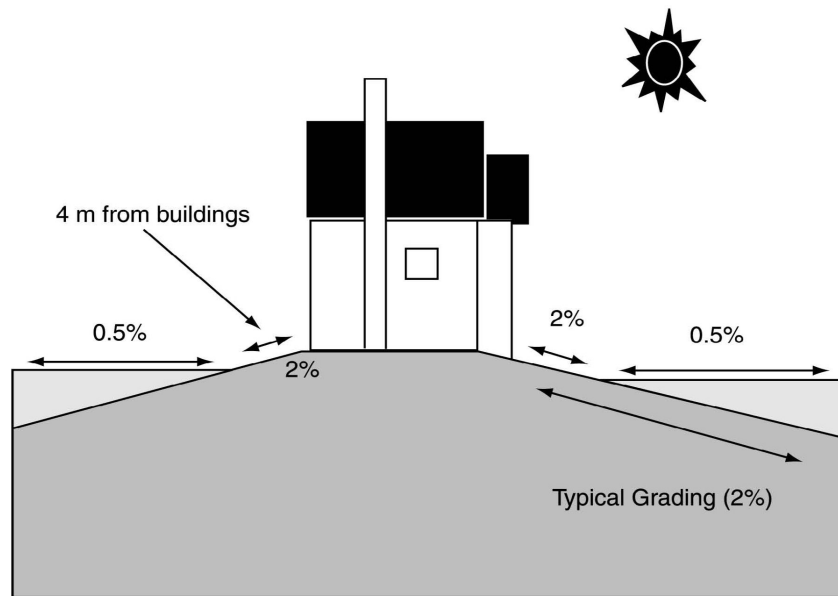


Figure 5.1
Lot Grading Guidelines

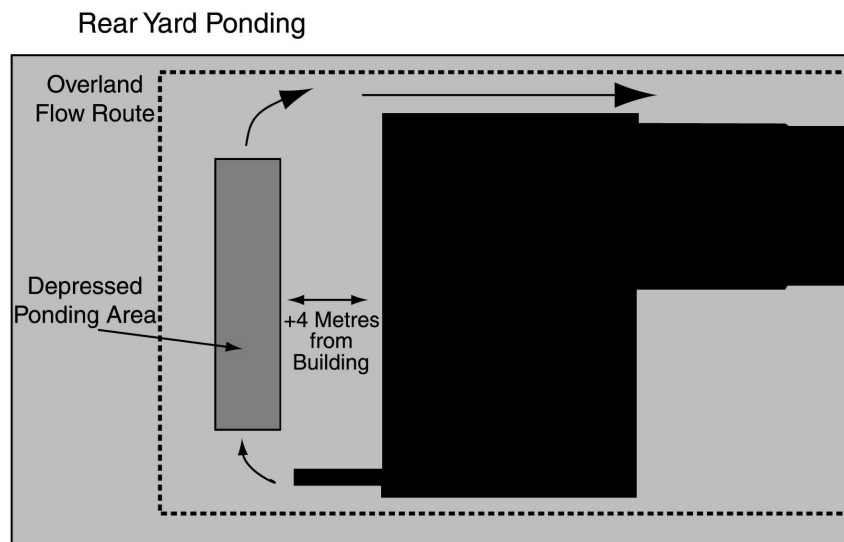


Figure 5.2
Rear Lot Ponding Guidelines

5-41

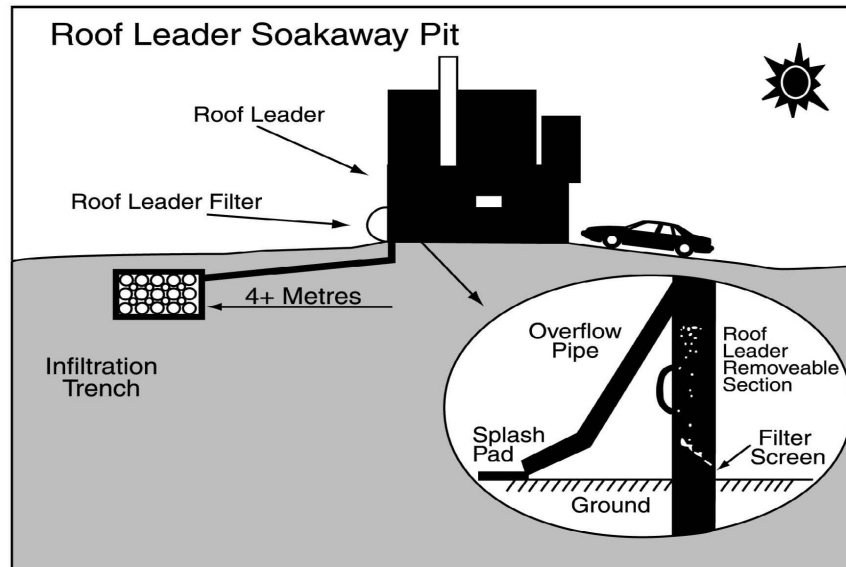


Figure 5.3
Infiltration System with Roof Leader Filter

5-42

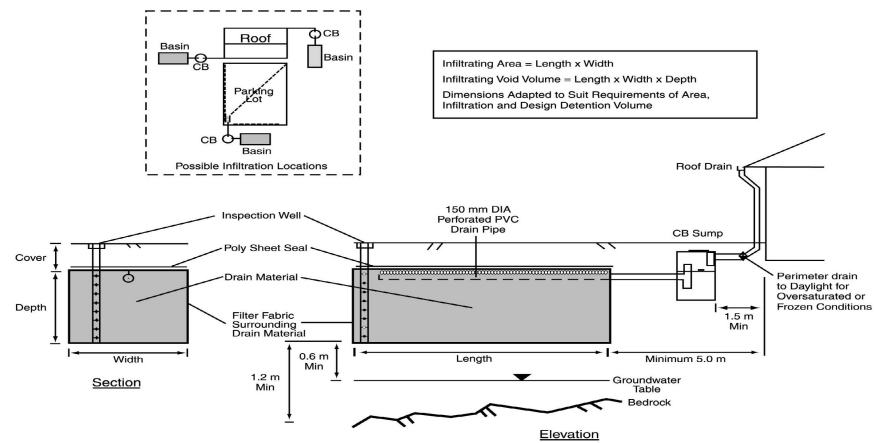


Figure 5.4
Infiltration System with Pretreatment Sump

5-43

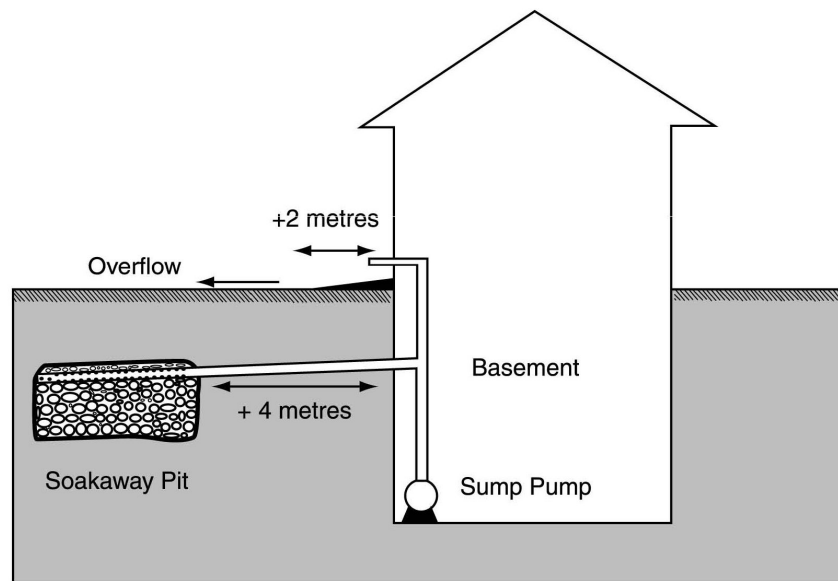


Figure 5.5
Sump Pump Foundation Drainage

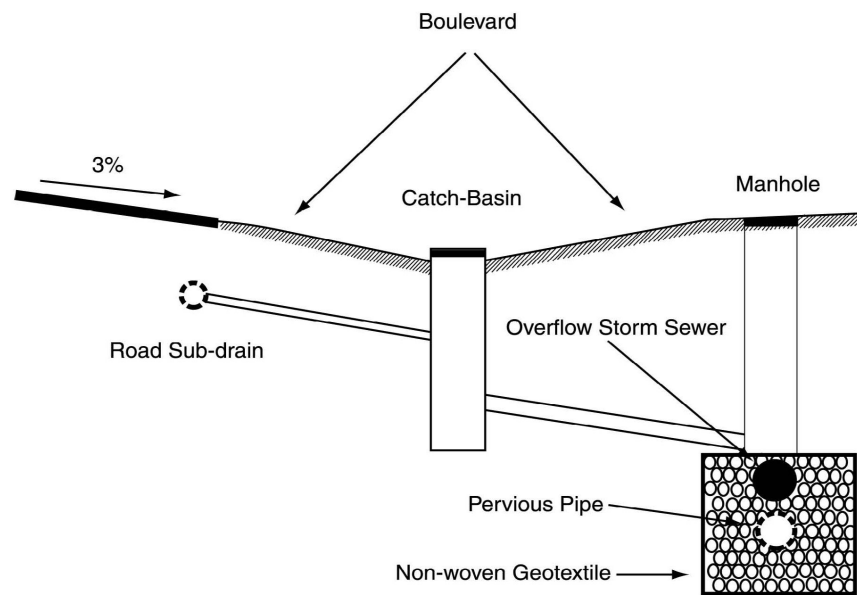


Figure 5.6
Pervious Pipe System

5-45

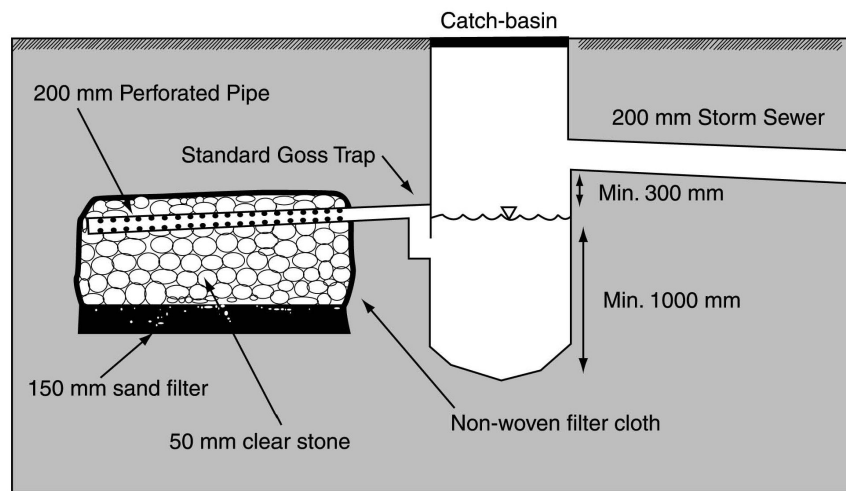


Figure 5.7
Pervious Catch-Basin

5-46

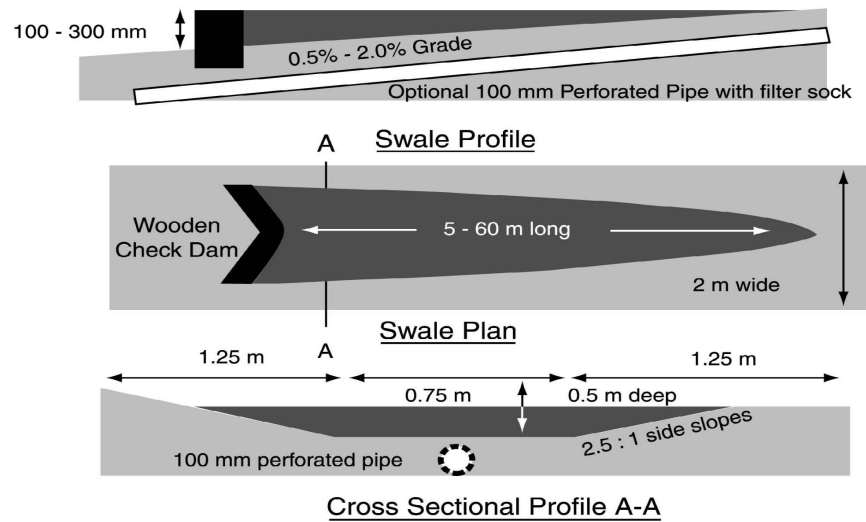


Figure 5.8
Grass Swale Design

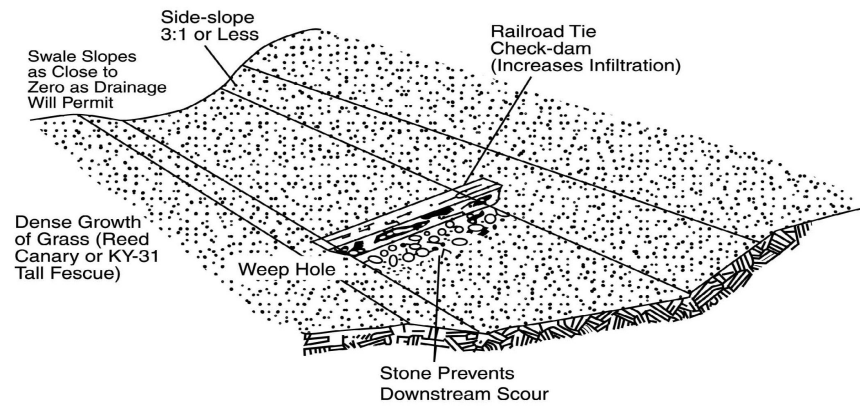
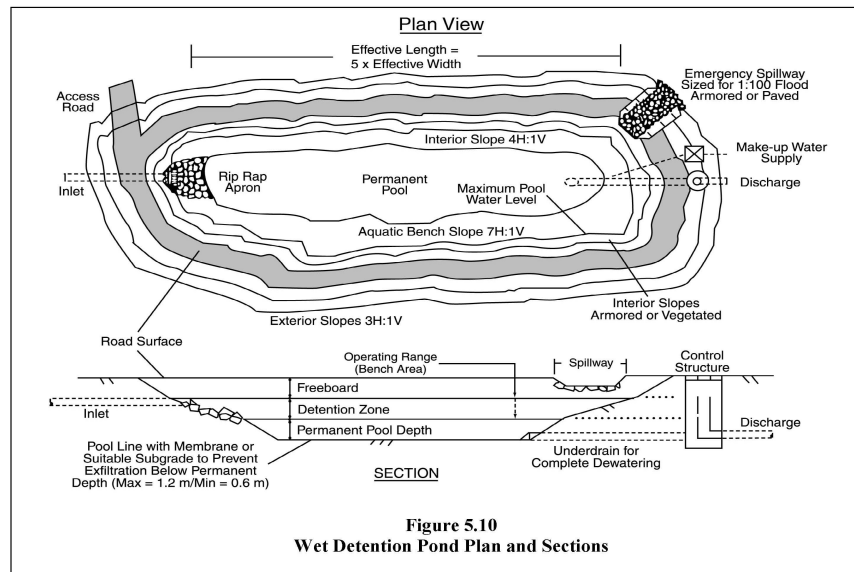


Figure 5.9
Grass Swale with Check Dam



5-49

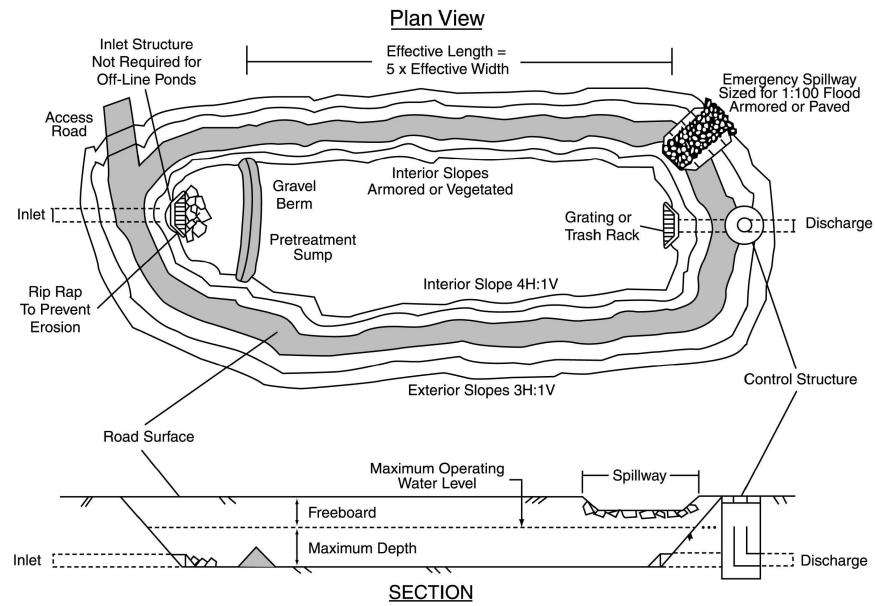
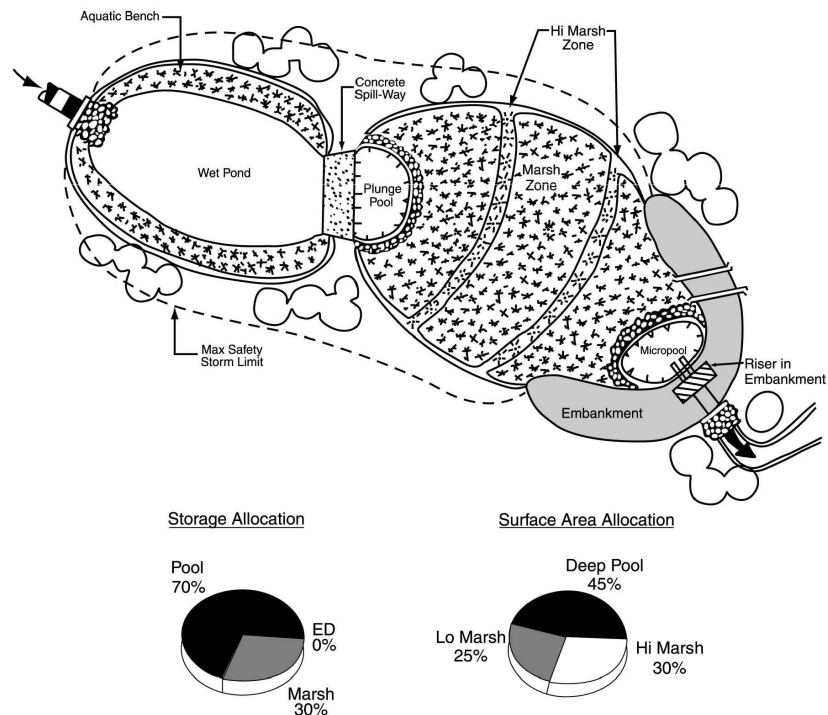


Figure 5.11
Dry Detention Pond Plan and Sections

5-50



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 5.12
Stormwater Wetland

5-51

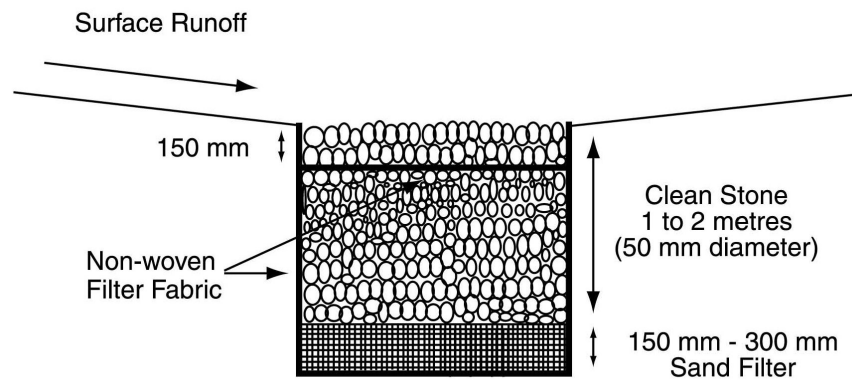


Figure 5.13
Surface Infiltration Trench

5-52

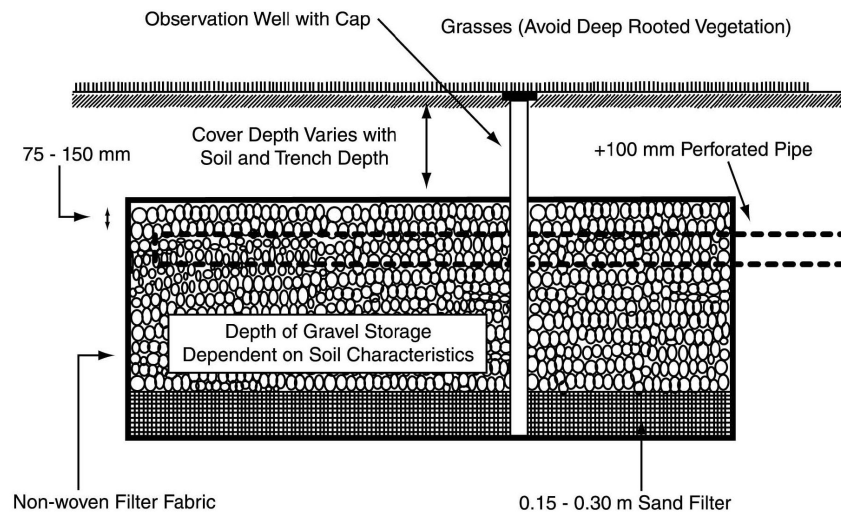


Figure 5.14
Subsurface Infiltration Trench

5-53

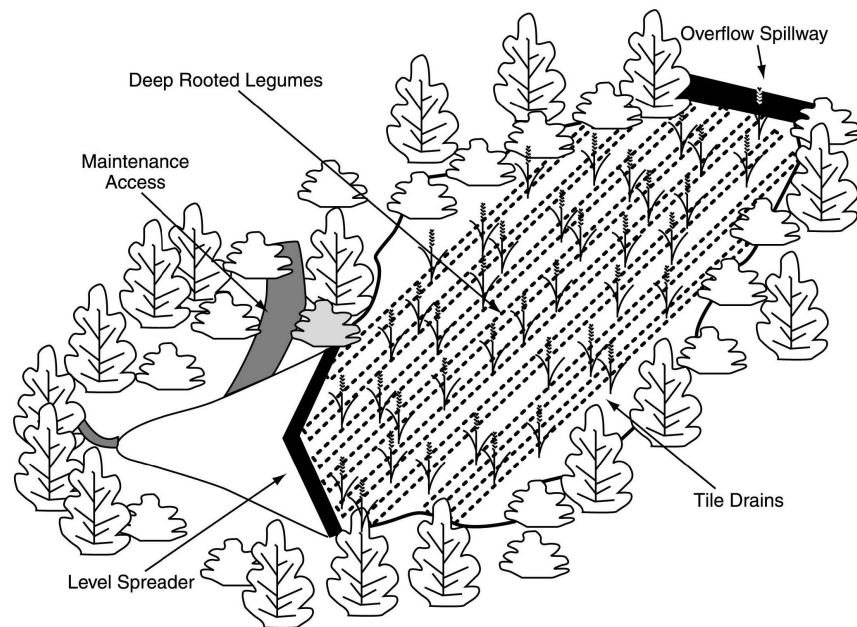


Figure 5.15
Infiltration Basin

5-54

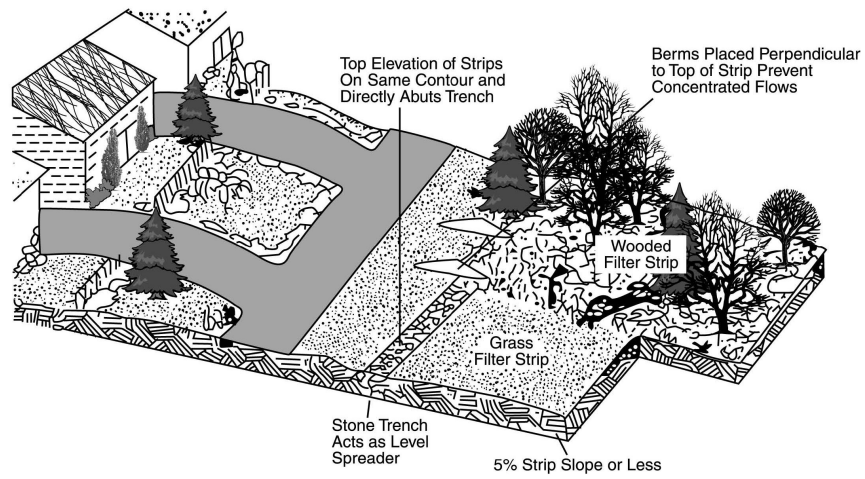


Figure 5.16
Schematic of Grassed and Wooded Filter Strip

5-55

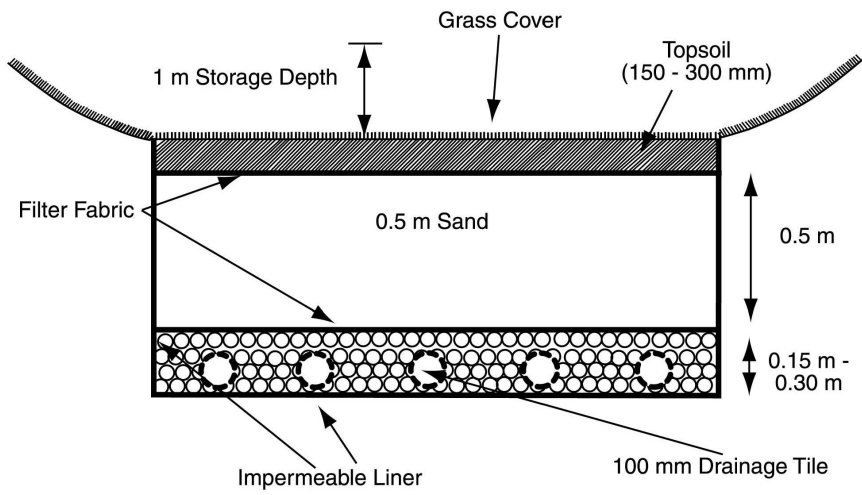


Figure 5.17
Sand Filter Cross Section Profile

5-56

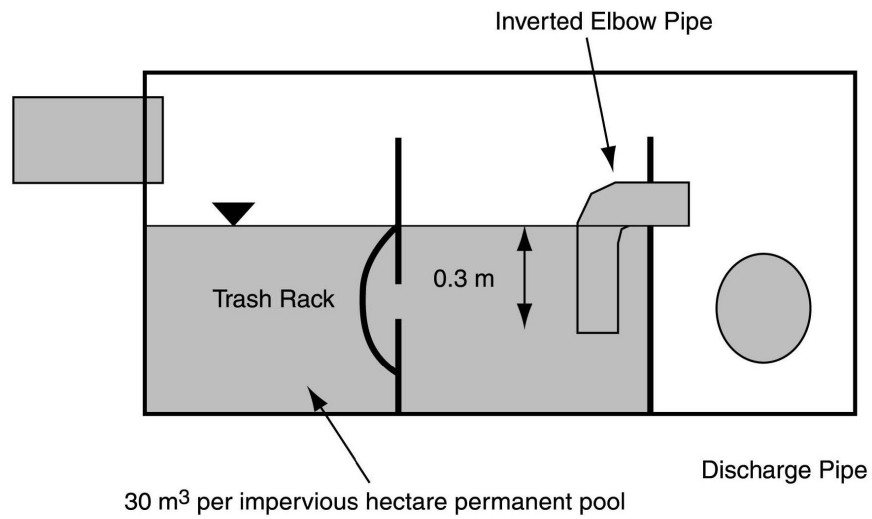


Figure 5.18
Standard 3 Chamber Oil / Grit Separator

5-57

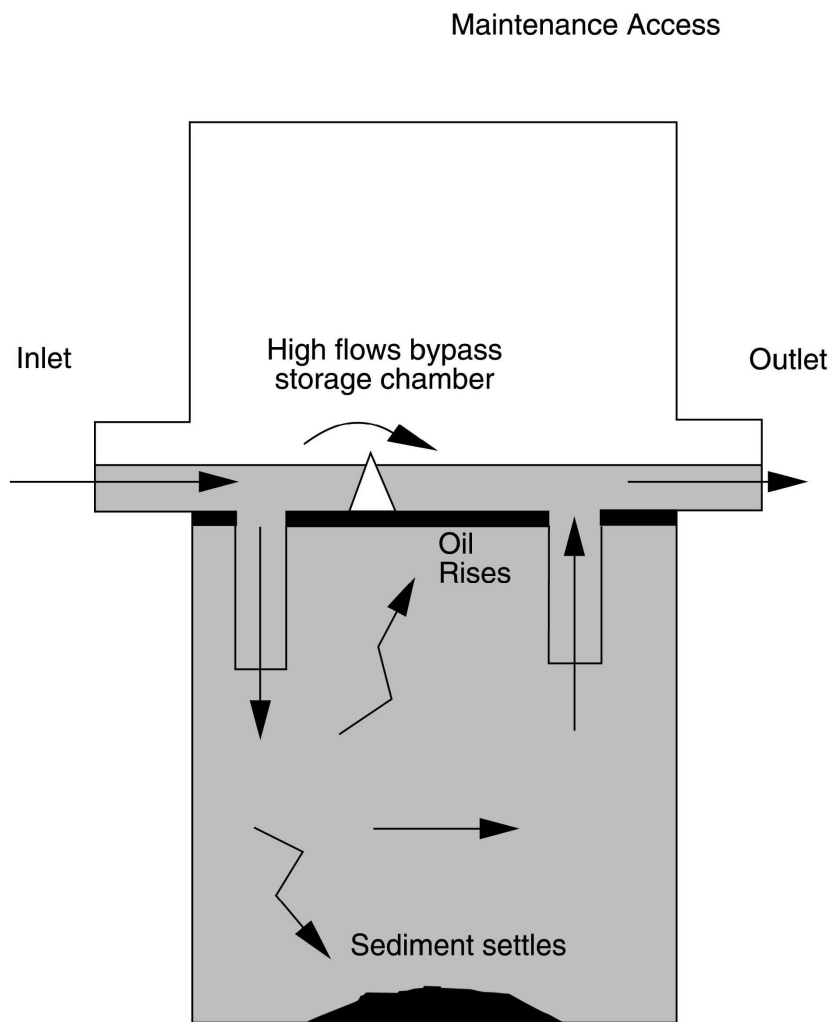


Figure 5.19
Bypass Separator

5-58

Justice and Solicitor General

**Designation of Qualified Technician Appointment
(Intoxilyzer 5000C)**

Calgary Police Service
Taylor, Jason James Ronald

(Date of Designation February 21, 2013)

**Office of the Public Trustee
Property being held by the Public Trustee for a period of Ten (10) Years
(Public Trustee Act)
Section 11 (2)(b)**

Name of Person Entitled to Property	Description of Property held and its value or estimated value	Property part of deceased person's	Public Trustee Office
		Estate or held under Court Order: Deceased's Name Judicial District Court File Number	
Stephen Clogg	Cash on hand \$1,448.35	Letters of Administration 1970 Judicial District of Calgary Surr 43482	Additional Information
Irma Johnson	Cash on hand \$1,871.18		
Estate of Ernest Gordon Edwards	Cash on hand \$13,140.89		
Phyllis Duke-Morgan	Share of estate	Alexander Morgan 143526 JD of Edmonton SES03 118820	

Municipal Affairs

Hosting Expenses Exceeding \$600.00 For the period October 1 to December 31, 2012

Function: *Municipal Government Act (MGA) Review Working Group Orientation Session*

Purpose: To serve as an introduction to the *MGA* Review project and to communicate the expectations of working group members. These working groups are comprised of over 140 Municipal Affairs staff involved in varying degrees in the *MGA* Review project.

Amount: \$10,446.83

Date of Function: September 13, 2012

Location: Edmonton

Function: 2012 Alberta Urban Municipalities Association (AUMA) Convention

Purpose: The Minister hosts an open house during the AUMA provincial convention for elected and administrative municipal and provincial officials, as well as other interested delegates.

Amount: \$17,114.97

Date of Function: September 26-28, 2012

Location: Edmonton

Function: Municipal Internship Program – Executive Week

Purpose: To provide interns with a better understanding of Municipal Affairs and other provincial ministries and help develop their professional network during their internship.

Amount: \$2,897.52

Date of Function: October 30 - November 2, 2012

Location: Edmonton

Function: Alberta Association of Municipal Districts and Counties (AAMD&C) Fall 2012 Convention – Minister's Open House

Purpose: To promote the relationship with elected officials in municipalities from across the province.

Amount: \$10,347.28

Date of Function: November 13, 2012

Location: Edmonton

Function: Alberta Emergency Management Agency (AEMA) 2012 Stakeholder Summit

Purpose: To bring together stakeholders from multiple disciplines in collaboration and discussion about emergency management in Alberta.

Amount: \$48,568.50

Date of Function: November 19-20, 2012

Location: Edmonton.

Safety Codes Council

Agency Accreditation - Cancellation

Pursuant to Section 30 of the Safety Codes Act it is hereby ordered that

Alberta Inspection Services, Accreditation No. A000115, Order No. 0115

Is to cease services under the Safety Codes Act for **Gas**

Consisting of all parts of the Natural Gas and Propane Installation Code and Propane Storage and Handling Code, including applicable Alberta amendments and regulations.

Issued Date: February 25, 2013.

Pursuant to Section 30 of the Safety Codes Act it is hereby ordered that

Alberta Inspection Services, Accreditation No. A000115, Order No. 0114

Is to cease services under the Safety Codes Act for **Plumbing**

Consisting of all parts of the National Plumbing Code and Alberta Private Sewage Systems Standard of Practice, including applicable Alberta amendments and regulations.

Issued Date: February 25, 2013.

Pursuant to Section 30 of the Safety Codes Act it is hereby ordered that

Hillside Home Inspection Services Inc., Accreditation No. A000103, Order No. 0106

Is to cease services under the Safety Codes Act for **Building**

Consisting of all parts of the Alberta Building Code including applicable Alberta amendments and regulations.

Issued Date: February 13, 2013.

Corporate Accreditation

(Safety Codes Act)

Pursuant to section 28 of the Safety Codes Act it is hereby ordered that

Baytex Energy Ltd., Accreditation No. C000187, Order No. 0723

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Corporation's industrial facilities identified in the approved Quality Management Plan for the discipline of **Electrical**

Consisting of all parts of the Canadian Electrical Code, Code for Electrical Installations at Oil & Gas Facilities and Alberta Electrical Utility Code. Including applicable Alberta amendments and regulations.

Accredited Date: February 22, 1996

Issued Date: February 27, 2013.

Corporate Accreditation – Amendment

(Safety Codes Act)

Pursuant to section 28 of the Safety Codes Act it is hereby ordered that

Direct Energy Resources, Accreditation No. C000260, Order No. 1565

Due to a change in scope and having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Corporation's industrial facilities identified in the approved Electrical Regulatory Management Plan for the discipline of **Electrical**

Consisting of all parts of the Canadian Electrical Code, Code for Electrical Installations at Oil & Gas Facilities and Alberta Electrical Utility Code.

Accredited Date: November 7, 2005

Issued Date: February 12, 2013.

Joint Municipal Accreditation - Amendment

(Safety Codes Act)

Pursuant to section 26 of the Safety Codes Act it is hereby ordered that

Municipal District of Bonnyville No 87, Town of Bonnyville, Village of Glendon, Summer Village of Bonnyville Beach, Summer Village of Pelican Narrows, Accreditation No. J000115, Order No. 0575

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Building**

Consisting of all parts of the Alberta Building Code including applicable Alberta amendments and regulations.

Accredited Date: December 21, 1995

Issued Date: February 13, 2013.

Pursuant to section 26 of the Safety Codes Act it is hereby ordered that

**Municipal District of Bonnyville No 87, Town of Bonnyville, Village of Glendon,
Summer Village of Bonnyville Beach, Summer Village of Pelican Narrows,**
Accreditation No. J000115, Order No. 0905

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Electrical**

Consisting of all parts of the Canadian Electrical Code, Code for Electrical Installations at Oil and Gas Facilities and Alberta Electrical Utility Code including applicable Alberta amendments and regulations.

Accredited Date: October 10, 1996

Issued Date: February 13, 2013.

Pursuant to section 26 of the Safety Codes Act it is hereby ordered that

**Municipal District of Bonnyville No 87, Town of Bonnyville, Village of Glendon,
Summer Village of Bonnyville Beach, Summer Village of Pelican Narrows,**
Accreditation No. J000115, Order No. 0776

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Gas**

Consisting of all parts of the Natural Gas and Propane Installation Code and Propane Storage and Handling Code, including applicable Alberta amendments and regulations.

Accredited Date: March 27, 1996

Issued Date: February 13, 2013.

Pursuant to section 26 of the Safety Codes Act it is hereby ordered that

**Municipal District of Bonnyville No 87, Town of Bonnyville, Village of Glendon,
Summer Village of Bonnyville Beach, Summer Village of Pelican Narrows,**
Accreditation No. J000115, Order No. 0775

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Plumbing**

Consisting of all parts of the National Plumbing Code and Alberta Private Sewage Systems Standard of Practice, including applicable Alberta amendments and regulations.

Accredited Date: March 27, 1996

Issued Date: February 13, 2013.

Municipal Accreditation
(Safety Codes Act)

Pursuant to Section 26 of the Safety Codes Act it is hereby ordered that

City of Cold Lake, Accreditation No. M000261, Order No. 2872

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Building**

Consisting of all parts of the Alberta Building Code including applicable Alberta amendments and regulations.

Accredited Date: December 21, 1995

Issued Date: February 13, 2013.

Pursuant to Section 26 of the Safety Codes Act it is hereby ordered that

City of Cold Lake, Accreditation No. M000261, Order No. 2873

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Electrical**

Consisting of all parts of the Canadian Electrical Code, Code for Electrical Installations at Oil and Gas Facilities including applicable Alberta amendments and regulations. Excluding the Alberta Electrical Utility Code.

Accredited Date: October 10, 1996

Issued Date: February 13, 2013.

Pursuant to Section 26 of the Safety Codes Act it is hereby ordered that

City of Cold Lake, Accreditation No. M000261, Order No. 2874

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Gas**

Consisting of all parts of the Natural Gas and Propane Installation Code and Propane Storage and Handling Code, including applicable Alberta amendments and regulations.

Accredited Date: March 27, 1996

Issued Date: February 13, 2013.

Pursuant to Section 26 of the Safety Codes Act it is hereby ordered that

City of Cold Lake, Accreditation No. M000261, Order No. 2875

Having satisfied the terms and conditions of the Safety Codes Council is authorized to administer the Safety Codes Act within the Municipality's boundaries in accordance with the approved Uniform Quality Management Plan for the discipline of **Plumbing**

Consisting of all parts of the National Plumbing Code and Alberta Private Sewage Systems Standard of Practice, including applicable Alberta amendments and regulations.

Accredited Date: March 27, 1996

Issued Date: February 13, 2013.

ADVERTISEMENTS

Notice of Certificate of Intent to Dissolve

(Business Corporations Act)

Notice is hereby given that a Certificate of Intent to Dissolve was issued to **498306 Alberta Ltd.** on December 20, 2012.

Dated at Calgary, Alberta on February 22, 2013.

Thomas J. Taylor, *Barrister and Solicitor.*

Notice is hereby given that a Certificate of Intent to Dissolve was issued to **2016181352 Pence Plus Projects Ltd.** on February 25, 2013.

Dated at Calgary, Alberta on February 26, 2013.

Sarah Jane Pentz, *President.*

Notice is hereby given that a Certificate of Intent to Dissolve was issued to **Demand Data Services Inc.** on February 28, 2013.

Dated at Calgary, Alberta on February 28, 2013.

Bryant D. Frydberg, *Solicitor.*

Public Sale of Land

(Municipal Government Act)

Rocky View County

Notice is hereby given that under the provisions of the Municipal Government Act, Rocky View County will offer for sale, by public auction, in the Municipal Office, 911 – 32 Avenue NE Calgary, Alberta, on Friday, May 3, 2013, at 2:00 p.m., the following lands:

Title	LINC #	Lot	Blk	Plan	Qtr	Sec	Twp	Rge	Mer	Roll #
011310617	0021591277				SE	35	22	28	04	02335012
101059899	0033800830	10	4	0911274	NE	14	23	27	04	03214210
061354743	0030362099	47	1	0410597	NE	22	23	27	04	03222375
061461708	0031446537	2	15	0514172	NE	22	23	27	04	03222461
061531065	0020119947	22-24	6	2319L	SW	23	23	27	04	03223063
071117576	0026564436	1	3	9512554	NW	23	23	27	04	03223251
061043222	0027329663	14	14	9810012	SW	23	23	27	04	03223502
971188045	0025668336	1		9311989	SE	31	23	27	04	03231017
061194827	0031637143	2	2	0611015	NW	04	23	28	04	03304016
091341506	0033084659	1	5	0811006	NW	04	23	28	04	03304018
071074238	0028438505	3	7	0011638	SE	05	23	28	04	03305015
971327804	0017578312	11		2821EZ	NW	12	23	05	05	03912082
111324581	0035051557				NW	12	23	05	05	03912218
961302541	0014648521		11	8410246	S	17	23	05	05	03917021
051140541	0028294833	Unit 6		0010219	SE	25	23	05	05	03925095
031229136	0029987550	2	1	0311954	SE	20	24	27	04	04220008
091013173	0029264059	5	1	0211010	NW	28	24	28	04	04328019
081071074	0029021748	1		0112710	SE	32	24	28	04	04332021
061382732	0031121809	17	14	0512238	NE	33	24	28	04	04333035
991279944	0027356930	Unit 17		9810245	SE	06	24	02	05	04606022
041259693	0029349081	Unit 66		0212033	NW	06	24	02	05	04606290
101233351	0021237185				NE	08	24	02	05	04608005

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071475261	0028969856	2B		8610826	NW	18	24	02	05	04618039
071550161	0032754757	Unit 44		0714894	N	02	24	03	05	04702135
011284230	0020412037		Pt 32	7910905	NE	03	24	03	05	04703006
091327335+8	0034087833	5	1	0914791	NE	14	24	03	05	04714004
061492357	0031682990	2	4	0611508	SW	14	24	03	05	04714074
061055275	0030586730	11	4	0412583	NW	08	25	28	04	05308020
031310380	0021591351				NE	12	25	28	04	05312005
061188071	0028498632	13		0012040	E	33	25	28	04	05333095
971120112	0026800459	Unit 35		9611742	SE	11	25	03	05	05711077
091084215	0029216835	Unit 4		0210521	SW	12	25	03	05	05712061
071493128	0030932768	1	1	0510531	NE	28	26	28	04	06328006
041154664	0027283150	2		9712387	SE	21	26	29	04	06421014
021442270	0028051878	18	1	9912375	W	22	26	29	04	06422049
101099801	0028478527	2	4	0011908	NE	28	26	29	04	06428056
851082141	0021239868				SW	10	26	02	05	06610004
081005474	0029358702	5	1	0212085	NE	1	26	03	05	06701102
021091373	0017069147	2		7910800	NE	11	26	03	05	06711013
051010814	0027848902	16	2	9910521	NE	12	26	03	05	06712059
061406938	0026503771	S1/21		9212319	SE	13	26	03	05	06713064
081017337+190	0032972432				NE	27	26	04	05	06827002
081017337+190	0032972424				NW	27	26	04	05	06827003
081017337+190	0019246438				SW	27	26	04	05	06827008
081017337+187	0032974313	Unit 177		0810165	NE	27	26	04	05	06827395
111035611	0032735384	1	1	0714772	SE	17	28	03	05	08717006
081163549	0032266538	4	1	0710865	NE	16	28	05	05	08916010

Each parcel will be offered for sale subject to a reserve bid and to the reservations and conditions contained in the existing Certificate of Title.

Rocky View County may, after the public auction, become the owner of any parcel of land that is not sold at the public auction.

Terms: CASH OR CERTIFIED CHEQUE

Deposit: 10% of bid at the time of the sale May 3, 2013

Balance: 90% of bid within 30 days of receipt by Rocky View County

Goods and Services Tax (GST) applicable as per Federal Statutes.

Redemption may be affected by payment of all arrears of taxes and costs at any time prior to the sale.

Dated at Calgary, Alberta, February 22, 2013.

Kent Robinson, *General Manager, Corporate Services.*

County of St. Paul No. 19

Notice is hereby given that, under the provisions of the Municipal Government Act, the County of St. Paul No. 19 will offer for sale, by public auction, in the County Office, St. Paul, Alberta, on Tuesday, May 14, 2013, at 1:30 p.m., the following lands:

Pt. of Sec.	Sec	Twp	Rge	Mer	Acres	C of T
NE	28	58	9	4	157.97	922075511
S ½ NW	15	58	10	4	79.70	942039991
N ½ SW	3	59	10	4	70.00	082103830+2
S ½ SW	3	59	10	4	80.00	082103830+1
SW	3	59	10	4	10.01	082103830
NW	34	57	11	4	158.00	992052775+2
NE	35	57	11	4	160.00	992052775+3
SW	4	58	11	4	158.01	882003160

Ashmont

Lot	Block	Plan	C of T
16-18	1	1379CL	042509432
8	5	2932HW	092328402+1
9	5	2932HW	072401442

Lottie Lake

Lot	Block	Plan	C of T
21	7	8121231	962337484

Lower Mann Lake

Lot	Block	Plan	C of T
10	1	6815MC	012002573+2
20	1	6815MC	092187929
25	1	3909TR	052402708

Lower Therien Lake

Lot	Block	Plan	C of T
69	2	7521646	962229613
203	5	7922205	062317731

Perch Lake

Lot	Block	Plan	C of T
37	2	8022307	992061416

St. Lina

Lot	Block	Plan	C of T
-	A	84HW	932210900

Upper Mann Lake

Lot	Block	Plan	C of T
36	1	7620518	102028381+2
37	1	7620818	102028381
38	1	7620518	102028381+1
5	2	8222114	092299576

Vincent Lake

Lot	Block	Plan	C of T
2	6	8121370	972137537
2	1	527MC	102389806
3	1	527MC	102389806+1

Each parcel will be offered for sale, subject to a reserve bid and to the reservations and conditions contained in the existing certificate of title.

These properties are being offered for sale on "as is" "where is" basis and the County of St. Paul No. 19 makes no representation and gives no warranty whatsoever as to the adequacy of services, soil conditions, land use districting, building and development conditions, absence or presence of environmental contamination, or the developability of the subject land for any intended use by the Purchaser.

The County of St. Paul No. 19 may, after the public auction, become the owner of any parcel of land not sold at the public auction.

All Bidders or their Agents must be present at the Public Auction.

Terms: Cash. The above properties may be subject to G.S.T.

Redemption may be affected by payment of all arrears of taxes and costs at any time prior to the sale.

Dated at St. Paul, Alberta, February 22, 2013.

Sheila Kitz, *Chief Administrator Officer*.

County of Wetaskiwin No. 10

Notice is hereby given that under the provisions of the Municipal Government Act, the County of Wetaskiwin No. 10 will offer for sale, by public auction, in the County Office at Wetaskiwin, Alberta, on Thursday, May 2, 2013, at 9:30 a.m., the following lands:

Roll #	Certificate of Title # or Linc #	Plan; Block; Lot	Mer	Rge	Twp	Sec	Parcel Size (Acres)
126901	072677933		4	24	47	14 NW	41.14
134502	052133830	7820660; 2; 32	4	2	47	34 NE	3.02
171202	072235486		4	25	47	36 SW	5.51
175300	872059264		4	26	45	9 NE	160.00

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175500	842269204		4	28	45	9 SE	158.00
196601	942039382	9320847; ;1	4	26	46	26 NW	8.90
233303	042010967		4	27	46	10 NE	.860
294906	762137333		5	1	46	12 NE	.210
296012	062453160	0520354; 1; 42	5	1	46	14 SE	.170
298900	062086519	0122727; ;2	5	1	46	22 NE	43.24
301000	032191630	0122173; ;1	5	1	46	31 NW	19.94
327900	962185886		5	2	47	5 SW	156.00
408500	052444823		5	5	46	5 NW	143.31
454305	072232641	0627064; 1; 5	5	6	46	32 SE	4.44
478830	112186898	1833KS; 7; 21&22	5	7	46	4 SE	.350
478913	062574150	3005KS; 2; 17	5	7	46	4 SW	.170
478949	062216309	6055HW; 2; 1	5	7	46	4 SW	.160

Each parcel will be offered for sale subject to a reserve bid and to the reservations and conditions contained in the existing Certificate of Title.

The County of Wetaskiwin No. 10 may, after the public auction, become the owner of any parcel of land that is not sold at the public auction.

Terms: 10% Cash Deposit and balance payable by cash or certified cheque within 48 hours.

Redemption may be affected by payment of all arrears of taxes and costs at any time prior to the sale.

Dated at Wetaskiwin, Alberta, February 22, 2013.

Rod Hawken, Assistant County Administrator.

Town of High River

Notice is hereby given that, under the provisions of the Municipal Government Act, the Town of High River will offer for sale, by public auction, in the Council Chambers, Town Administration Building, High River, Alberta, on Friday, May 3, 2013, at 10:00 a.m., the following lands:

Lot	Block	Plan	C. of T.	Civic Address
3	1	2254JK	071549304	95 2A St SE

This parcel will be offered for sale, subject to a reserve bid and to the reservations and conditions contained in the existing certificate of title.

The Town of High River may, after the public auction, become the owner of any parcel of land not sold at the public auction.

Terms: Cash, certified cheque or a letter of credit from a banking institution that must be submitted by the close of the sale.

The above listing is subject to change upon payment of all arrears of taxes and costs at any time prior to the sale.

Dated at High River, Alberta, February 20, 2013.

Tom Maier, *Chief Financial Officer*.

NOTICE TO ADVERTISERS

The Alberta Gazette is issued twice monthly, on the 15th and last day.

Notices and advertisements must be received ten full working days before the date of the issue in which the notices are to appear. Submissions received after that date will appear in the next regular issue.

Notices and advertisements should be typed or written legibly and on a sheet separate from the covering letter. An electronic submission by email or disk is preferred. Email submissions may be sent to the Editor of The Alberta Gazette at albertagazette@gov.ab.ca. The number of insertions required should be specified and the names of all signing officers typed or printed. Please include name and complete contact information of the individual submitting the notice or advertisement.

Proof of Publication: Statutory Declaration is available upon request.

A copy of the page containing the notice or advertisement will be mailed to each advertiser without charge.

The dates for publication of Tax Sale Notices in The Alberta Gazette are as follows:

<i>Issue of</i>	<i>Earliest date on which sale may be held</i>
March 30	May 10
April 15	May 26
April 30	June 10
May 15	June 25
May 31	July 11
June 15	July 26
June 29	August 9
July 15	August 25
July 31	September 10
August 15	September 25
August 31	October 11
September 14	October 25

The charges to be paid for the publication of notices, advertisements and documents in The Alberta Gazette are:

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